FRA - 82 - 3

U.S. Department of Transportation

Federal Railroad Administration

Ballast and Subgrade Requirements Study

Office of Research and Development
Washington, DC 20590

Railroad Track Substructure - Materials Evaluation and Stabilization Practices

R.M. Simon
L. Edgers
J.V. Errico

Goldberg-Zoino & Associates, Inc.
30 Tower Road
Newton Upper Falls, Massachusetts 02164

FRA/ORD-83/04.1

June 1983
Final Report

This document is available to the U.S. Public through the National Technical Information Service, Springfield, Virginia 22161.

Technical Report Documentation Page

1. Report No. FRA/ORD-83/04.1	2. Government Accession No.	3. Recipient's Catalog No.
4. Title and Subtitle BALLAST AND SUBGRADE REQUIREMENTS STUDY: RAILROAD TRACK SUBSTRUCTURE - MATERIALS EVALUATION AND STABILIZATION PRACTICES		5. Report Date June 1983
		6. Performing Organization Code DTS-73
7. Author(s) Simon, R.M., L. Edgers, and J.V. Errico		8. Performing Organization Report No. DOT-TSC-FRA-82-3
9. Performing Organization Name and Address Goldberg-Zoino & Associates, Inc. * 30 Tower Road Newton Upper Falls, Massachusetts 02164		10. Work Unit No. (TRAIS) RR119/R1320
		11. Contract or Grant No. DOT-TSC-1527
12. Sponsoring Agency Name and Address U.S. Department of Transportation Federal Railroad Administration Office of Research and Development Washington, D.C. 20590		13. Type of Report and Period Covered Final Report July 1978-January 1981
		14. Sponsoring Agency Code RRD-10

15. Supplementary Notes
*Under contract to: U.S. Department of Transportation
Research and Special Programs Administration
Transportation Systems Center
Cambridge, Massachusetts 02142

16. Abstract

Earth materials--i.e., soil and rock--form the substructure of all railroad track. In this report, the functions and performance characteristics of each of the substructure elements (i.e., ballast, subballast, and subgrade), and the material properties that influence the substructure performance are described. In addition, guidelines are provided for their use in railroad track.

A primary emphasis has been placed on the use of index property tests correlated with engineering property tests to characterize earth materials for railroad track engineering. Mechanical, environmental, permeability, and construction performance characteristics are discussed, and recommendations are provided for the exploration, characterization, testing, and classification of earth materials--both for proposed track use and for observing in-track substructure materials. We have also provided recommended practices that will generate data for future evaluations of earth materials in railroad track.

Subgrade stabilization methods are discussed. These are procedures that may be used to improve the performance characteristics of subgrade soils and to reduce maintenance requirements of existing or proposed track and/or to permit upgrading of track loading while limiting future track displacements. Emphasis is placed on methods that can treat the subgrade with limited disruption of train operations.

17. Key Words Railroad Track, Ballast, Subgrade, Subballast, Substructure, Field Tests, Laboratory Tests, Performance		18. Distribution Statement Document is available to the public through the National Technical Information Service, Springfield, Virginia 22161	
19. Security Classif. (of this report) Unclassified	20. Security Classif. (of this page) Unclassified	21. No. of Pages 386	22. Price

Form DOT F 1700.7 (8-72) Reproduction of completed page authorized

PREFACE

This work is part of a study of railroad ballast and subgrade requirements including synthesis of track substructure materials engineering and stabilization practices, and practices for the design of the substructure for conventional railroad tracks. This report concerns use of earth materials in the substructure of conventional railroad track and stabilization of track subgrades. The study was conducted by Goldberg-Zoino & Associates, Inc., (GZA) of Newton Upper Falls, Massachusetts, for the U.S. Department of Transportation's Transportation Systems Center (TSC) in Cambridge, Massachusetts, under Contract DOT-TSC-1527, and was sponsored by the Federal Railroad Administration (FRA), Office of Rail Safety Research, Improved Track Structures Research Division, Washington, D.C.

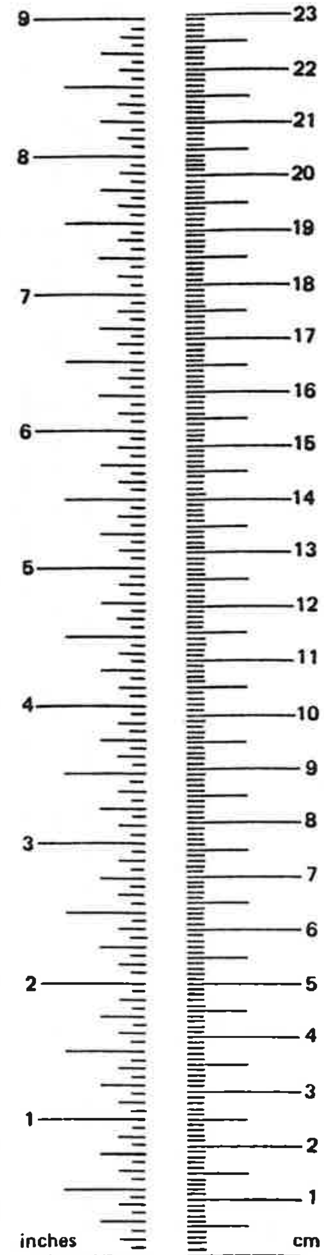
The TSC Technical Monitor for this project is Mr. James Lamond. Mr. Andrew Sluz of TSC also provided substantial technical guidance during the study. The Principal Investigator for the study was Dr. Richard M. Simon, Senior Geotechnical Engineer at GZA. Dr. Lewis Edgers of the Civil Engineering Department, Tufts University, contributed to the material on subgrade soils and reviewed the report. Mr. James V. Errico of GZA headed the study of subballast. Messrs. Peter K. Hadley and M. Daniel Gordon contributed to the section on substructure stabilization methods. Mr. Lionel Peckover, Geotechnical Consultant, of Quebec, Canada, and Mr. J. B. Farris of the Southern Railway and Mr. K. F. Briggs of the Boston and Maine Railroad provided valuable consulting input on ballast and geotechnical engineering practices by operating railroads. Personnel from the firm of Thomas K. Dyer, Inc., Lexington, Massachusetts, cooperated with us in development of this report. Mr. Donald T. Goldberg, GZA, contributed significantly to the chapter on subgrade soils and served as overall project reviewer. Ms. Donna Meeker conducted an initial survey of the literature. Ms. Susan Regenbogen Rosinoff was the project's Technical Editor. Ms. Donna Comeau prepared the final typed documents.

METRIC CONVERSION FACTORS

Approximate Conversions to Metric Measures

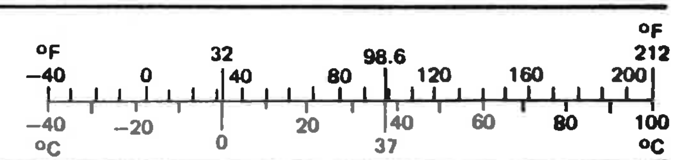
Symbol	When You Know	Multiply by	To Find	Symbol
		LENGTH		
in	inches	*2.5	centimeters	cm
ft	feet	30	centimeters	cm
yd	yards	0.9	meters	m
mi	miles	1.6	kilometers	km
		AREA		
in^2	square inches	6.5	square centimeters	cm^2
ft^2	square feet	0.09	square meters	m^2
yd^2	square yards	0.8	square meters	m^2
mi^2	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 cm (exactly). For other exact conversions and more detail tables see NBS Misc. Publ. 286. Units of Weight 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	36	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



EXECUTIVE SUMMARY

Objective: The objective of the study is to review and synthesize the best available technology that might be applied to design, construction, maintenance, and upgrading of conventional railroad track substructures. The first phase of the study develops suggested practices for exploring, testing, classifying, and selecting earth materials for use in railroad track substructures--i.e., ballast, subballast, and subgrades. The second phase identifies available technologies for stabilizing and improving the performance level of track subgrade soils, either to meet present loading demands with lowered maintenance requirements or to upgrade track to meet demands of higher axle loads or more stringent operating criteria. The final phase surveys available methods for analysis, design, and performance evaluation of track substructures. Design and performance evaluation will not be discussed herein.

Scope of Study: The scope of this study has been to review available technology in the railroad engineering field, as well as technologies in highway and airfield pavement engineering, geology, foundation engineering and related areas that can be applied directly to railroad substructure engineering. This review has included a broad survey of published literature, personal communications with practicing railroad engineers and researchers, and our own general expertise in dealing with earth materials in civil engineering construction. In the development of suggested practices, emphasis has been placed on easily performed, repeatable tests that can be economically carried out. Engineering property measurements are recommended where the parameters can be used reliably in analysis or engineering evaluation. Complex testing, such as cyclic triaxial testing of soils and ballast, has been deemphasized because it is believed that the cost of testing is not justified by the variability in results obtained and the natural variability of earth materials themselves.

The review of substructure stabilization included methods to treat subgrade soils to upgrade the performance characteristics of existing track in order to reduce maintenance requirements and to upgrade substructure performance to handle higher axle loads and greater traffic frequency and speed. Subgrade stabilization methods may be implemented for new construction, track rehabilitation, or as a part of regular track maintenance. Emphasis is placed on those methods that offer the potential for improved substructure performance while requiring limited disruption of the track and train operations.

Research Justification: Many American railroads have been beset by financial difficulties. A major factor compounding the plight of many railroads' financial problems is the escalating cost of maintenance. As costs and need for maintenance increase, it has been difficult to expand the maintenance funds to match the need, leading to accelerated deterioration of the track structure. Generally in the railroad industry, a deficit of maintenance

performed compared to maintenance needed but not performed is expanding, and the serviceability of railroad tracks is decreased. This is at a time when the demand for rail transport is increasing, such as for freight service and coal hauling. There is a trend toward higher axle loads that demand greater track strength and stability. There is the need to optimize the application of maintenance funds to counter the trend of increased costs, tighter maintenance budgets, and a demand for track safety and operating efficiency. To meet the goal of optimizing maintenance expenditures, this study has developed suggested practices for exploring, testing, classifying, and selecting earth materials for use in track substructure. Subgrade stabilization procedures described may be used to upgrade subgrade performance and to reduce maintenance requirements. The best available stabilization technologies are identified, and guidelines are provided for their application to railroad substructure improvement.

Summary of Results: Subgrade soils are the natural earth materials that form the base of the track substructure. Since the subgrade is determined by the location of the track route, the first step in subgrade engineering is to explore the subsurface to determine the nature of the subgrade materials. Subsurface exploration typically is done by test borings and test pits or trench excavations. These direct exploration methods may be supplemented by geophysical techniques, such as seismic refraction or electrical resistivity surveys, that can be used to determine the depth of soft soils, the position of the water table, and the top of rock or other stiff layer.

Exploration requirements in areas of cuts and areas of fills differ. Exploration should be spaced about 150m (500 feet) apart generally for both cuts and fills. Depth of explorations for cuts should be about 1.5 times the depth of the cut. For fills, the depth of explorations should be at least equal to the width of the fill. Explorations should penetrate weak, soft soils. Extra explorations are required in areas of potential embankment or slope instability or excessive settlement.

The performance of all substructure materials, including subgrades, may be described in terms of the following performance classes:

a. Mechanical--related to the ability of the materials to support the track and resist loads while maintaining acceptably small displacements. Mechanical properties include shear strength, stiffness or resilience, and long term compression or residual deformation properties.

b. Environmental--related to the resistance of materials to changes from factors such as freeze-thaw and moisture changes.

c. Permeability--related to the passage of water and migration of soil particles through the material.

d. Construction/Maintenance--related to the workability of the material and operation of equipment during construction and maintenance operations.

In engineering and selecting materials for use in the track substructure, each of these classes of performance must be considered. It should be recognized that the different factors interact. In particular, mechanical performance is generally affected by the other performance factors.

To design track substructures, an engineer should have a general appreciation of typical soil behavior, such as the factors that control shear strength; soil volume change processes, such as consolidation, swell, and collapse; and cyclic loading behavior. The Mohr-Coulomb failure criterion and the principle of effective stress for soils are pertinent.

To evaluate the properties of subgrade soils, laboratory engineering property tests, laboratory index tests, and in-situ or field tests may be used. Of the multitude of tests available, the following are judged to be of greatest value for railroad substructure engineering:

a. Laboratory index test

   1. Visual manual soil description
   2. Percentage finer than No. 200 sieve
   3. Grain size analysis
   4. Moisture content
   5. Atterberg limits
   6. Unconfined or unconsolidated-undrained triaxial shear strength

b. Field tests

   1. Standard penetration test
   2. Static (Dutch) cone penetration test
   3. Field vane shear test
   4. Plate bearing test

c. Laboratory Engineering tests (as required on a site specific basis)

   1. Consolidation test
   2. Consolidation-drained or -undrained triaxial test

In order to transfer substructure engineering practice from one locale to another, it is necessary to describe the subgrade soil properties in an unambiguous way. The Unified Soil Classification system is suggested for classifying subgrade soils.

In some cases, track conditions deteriorate at an unusually rapid rate. Sometimes, the cause of track performance deficiencies, such as loss of line and surface, tie rotting, and even track hardware fatigue, is deficient subgrade performance. The most prevalent subgrade problems develop from conditions close to the top of the substructure, such as mud pumping, ballast pockets, subgrade squeezes, swelling clays, frost heaves, collapsing soils, and embankment surface sloughs. Though affecting less track mileage, deep-seated, major foundation problems, such as embankment slides, creep, and consolidation under embankment load, can be continually troubling and sometimes cause complete interruption of track service. The subgrade deficiencies may be corrected by stabilization procedures described below.

Ballast performance has received a great deal of attention in the past 10 years. Studies have been carried out using various types of laboratory static and cyclic shear devices. Some full-scale track model tests have studied the effects of cyclic loading on ballast breakdown. A few programs have included systematic evaluation of ballast performance in service track. Generally, these studies have concentrated on the mechanical performance of ballast. Only limited study of environmental, permeability, and maintenance performance was discovered. The mechanical studies have pointed out the significance of particle hardness, toughness, shape, and angularity on the strength and stiffness of ballast. Confining stress level and shear stress level are also important factors determining resilient and residual stress-strain behavior of ballast.

Environmental factors have received some attention in the study of pavement aggregates but only limited attention in the railroad field. Freeze-thaw and general chemical mineral alteration are the principal factors that affect ballast performance. The permeability characteristic of ballast of primary significance relates to the movement of fine particles through the ballast bed. This factor is determined by ballast gradation. A broader particle gradation range (less uniform size) might improve ballast resistance to mud pumping from below and fouling of ballast from fines dropped on the surface. However, this hypothesis has had insufficient testing in track to determine its validity. Further study of optimum ballast gradation is warranted.

A great number of laboratory tests may provide indices of potential ballast performance in track. Thirteen have been selected in this study and are suggested as the appropriate tests for selecting and evaluating potential ballast sources. The tests are petrographic analysis, bulk specific gravity and water absorption, grain specific gravity, Los Angeles and mill abrasion tests, point load compressive strength, magnesium sulphate soundness, reference density, flakiness and elongation indices, sieve analysis, static crushing value, and the cementing value test. Definitive limits of parameter values for acceptable ballast have not been established for all these tests. However, if the test parameters are determined for ballasts that are observed to perform both well and poorly in track, it is anticipated that a reliable ballast testing/selection procedure can be developed in the future.

The principal functions of subballast are to separate the ballast and subgrade, while distributing train loads. The subballast may also serve to limit infiltration of surface water into the subgrade. These functions are influenced principally by particle size gradation characteristics. Subballast should have a small amount of fines (material finer the No. 200 sieve, 2 to 10 percent by weight) and should have a gradation related to the particle sizes of both the ballast and subgrade. Suggested gradation criteria similar to the criteria developed for graded aggregate filters, are presented in the report.

Substructure Stabilization refers to measures that treat subgrade soils to improve their performance characteristics. These measures may be applied to new construction, track rehabilitation, or to treat the substructure of in-service track.

Excess water aggravates all types of subgrade soil problems. Improving drainage measures is often the most cost-effective method of substructure stabilization. Drainage is of particular importance for track in cuts and in flat topography. Drainage problems can even develop in substructure of elevated track due to settlements and development of ballast pockets that destroy the proper grading profile of the subgrade surface.

The most common type of drain used in railroads is the lateral open ditch drain. This is used to carry surface runoff, and if deep enough, to control groundwater level. Interceptor drains, either open or as buried pipe or French drains, are important in controlling water flow in slopes. The principal difficulty in buried drains is to prevent movement of fine soil particles. This can be accomplished by providing a filter of properly graded aggregate or plastic filter fabric.

Some methods are available to stabilize subgrade soils in-place. Grouting with sand and/or Portland cement slurries have been used to stabilize slides in railroad embankments and to halt the progress of ballast pockets. The principal benefit is to limit the access of water to the soil.

Lime slurry pressure injection (LSPI) has been tried in recent years to stabilize soft clay subgrades. The lime slurry is injected through pipes that are inserted through the ballast. The intent is to reduce clay plasticity by the chemical reaction with the lime. It is difficult to imagine that sufficient lime can be injected into the soil to achieve significant improvement. Experience indicates that erratic improvement is realized.

Electrochemical stabilization involves the injection and dispersement of chemicals into clay soils with the aid of electrical currents. This method might be used effectively to treat subgrade soil beneath operating track; however, the cost is typically very high.

Application of rock salt to the surface of track has been studied by the Canadian National Railroad as a method to control frost heave. The method has been shown to be economical, simple, and 70 percent to 80 percent effective.

Layer inserts have become increasingly popular in track reconstruction to upgrade substructure performance. Subballast is the most commonly used insert and was discussed previously. Filter fabric or geotextiles have been used increasingly. The fabric provides a means to permit water movement, yet precludes the passage of fine soil particles. Experience with fabrics has seen both success and failure. Application criteria must be developed, such as, (1) do not place fabric directly on pure clay and silt subgrades without a sand blanket, and (2) there must be at least 6 inches, and preferably 8 inches to 12 inches, of ballast between the fabric and the base of the ties.

Compaction of the subgrade is probably the easiest way to improve subgrade soil properties for new construction or rebuilding. Admixtures may be added to soil to improve properties. Cement, lime, and bitumen are the most common additives. These materials can improve shear strength, decrease permeability, improve resistance to traffic disturbance, and improve workability during construction.

A few methods are available to stabilize movements due to deep-seated failures. Pile or pole driving attempts to stabilize movements by transferring support of the sliding soil to the piles. This type of treatment is sometimes adopted as an emergency remedial measure.

More permanent slope stabilization can be effected by adding berms or lowering slope heights. Engineered retaining structures, such as crib walls or cast-in-place retaining walls, can be used to arrest movements, albeit at significant costs.

Many other methods of stabilization are available for railroad applications. These generally require complete disruption of the track and are therefore only applicable to new construction or a complete track rebuild. The primary requirement for successful substructure stabilization is first to develop a clear understanding of the mechanisms that are causing substructure displacement. The available stabilization methods then may be selected with confidence to treat the causes of the problems.

TABLE OF CONTENTS

TABLE OF CONTENTS (Cont'd.)

LIST OF ILLUSTRATIONS

LIST OF ILLUSTRATIONS (Cont'd.)

LIST OF ILLUSTRATIONS (Cont'd.)

LIST OF ILLUSTRATIONS (Cont'd.)

LIST OF TABLES

LIST OF TABLES (Cont'd.)

LIST OF SYMBOLS

A	=	absorption
A	=	area of cross section normal to fluid flow
$\overline{A}_i$, $\overline{A}$	=	individual gradation modulus, gradation modulus
AASHTO	=	American Association of State Highway and Transportation Officials
AREA	=	American Railway Engineering Association
ASTM	=	American Society for Testing and Materials
B	=	bulk specific gravity
C	=	compression index for one-dimensional strain
°C	=	degrees celsius
C_c	=	coefficient of curvature (gradation); compression index for normally consolidated soil
C_r	=	compression index for overconsolidated soil
C_s	=	swell index for soil
C_u	=	coefficient of uniformity (gradation)
CBR	=	California bearing ratio
CD	=	consolidated-drained (triaxial test)
CPT	=	cone penetration test
CSR	=	coefficient of subgrade reaction
CU	=	consolidated-undrained (triaxial test)
D	=	free fall distance of hammer (compaction test)
D_n	=	sieve size through which n percent of the soil weight that is finer will pass
D_r	=	relative density
E	=	compacting energy; Young's modulus
FAA	=	Federal Aviation Administration
G	=	shear modulus
G_s	=	grain specific gravity
H	=	thickness of soil layer
LAA	=	Los Angeles abrasion
M	=	mass of hammer; Mega- (prefix)
M_s	=	mass of sample
N	=	number of blows per layer; standard penetration resistance
OCR	=	overconsolidation ratio
O.D.	=	outside diameter
Pa	=	pascal (= newton per square metre)
Q	=	quantity of seepage in a cross section
Q	=	quick test (unconsolidated-undrained triaxial test)
R	=	consolidated-drained (triaxial test)
S	=	degree of saturation
S	=	slow test (consolidated-drained triaxial test)
S_d	=	drained shear strength
S_t	=	sensitivity
S_u	=	undrained shear strength
SPT	=	standard penetration test

LIST OF SYMBOLS (CONTINUED)

T	=	time factor (consolidation)
USC	=	Unified Soil Classification
UU	=	unconsolidated undrained (triaxial test)
V_{cwr}	=	volume of container by water replacement method
c	=	cohesion intercept (Mohr-Coulomb failure model)
cm	=	centimetres
d, D	=	sieve size or particle size
d_i, $\overline{d}_i$	=	effective mean diameter — individual, weighted average
e	=	void ratio
e_{max}	=	void ratio of soil in loosest condition
e_{min}	=	void ratio of soil in densest condition
e_o	=	initial void ratio
Δe	=	change in void ratio
g	=	acceleration of gravity
i	=	hydraulic gradient; initial (subscript)
k	=	coeffecient of permeability; kilo- (prefix)
$\log_{10}$	=	common logarithm
$\log_e$	=	natural logarithm
m	=	meters
mm	=	millimeters
n	=	Talbot empirical exponent factor; total porosity
p	=	percentage
q_c	=	cone point resistance
q_s	=	cone sleeve resistance
q_u	=	unconfined compression strength
sec	=	seconds
t	=	time
u	=	pore water pressure
γ_d	=	dry density
γ_{ult}	=	ultimate unit weight (reference density)
γ_w	=	unit weight of water
γ_{wr}	=	unit weight of ballast by water replacement method
ρ	=	settlement
σ	=	total normal stress
$\overline{\sigma}$	=	effective normal stress
$\overline{\sigma}_{ff}$	=	effective normal stress acting on failure plane at failure
$\overline{\sigma}_{vf}$	=	final vertical effective stress

LIST OF SYMBOLS (CONTINUED)

$\bar{\sigma}_{vm}$	=	maximum past vertical effective stress
$\bar{\sigma}_{vo}$	=	initial vertical effective stress
τ	=	shear stress
τ_{ff}	=	shear stress acting on failure plane at failure
ϕ	=	angle of internal friction
$\bar{\phi}$	=	effective stress angle of internal friction
$\propto$	=	proportional to

1. INTRODUCTION

The Transportation Systems Center, (TSC), U.S. Department of Transportation, has been conducting track research for the Federal Railroad Administration's (FRA) Office of Rail Safety Research. This research program is aimed at improving the safety of rail service in the United States. Among the major goals are the reduction of track caused accidents and development of guidelines and specifications for improving the performance of track structures.

A major component of the railroad track system is the track substructure (i.e., the ballast, subballast, subgrade, and foundation). Within the past decade, a principal focus of the FRA's program of substructure research has been the analytic and empirical means of developing pertinent substructure design criteria. Under a contract with the Association of American Railroads sponsored by the FRA, workers in the Department of Civil Engineering at the University of Illinois in Urbana carried out an extensive research program including the development of a mathematical model of track and track substructure combined with performance testing of ballast and subgrade materials. Earth materials testing to date has concentrated on laboratory cyclic triaxial testing of earth materials and associated laboratory index tests.

The State University of New York at Buffalo and the University of Massachusetts in Amherst investigated the mechanics of ballast compaction and the in-situ measurement of ballast physical state; as part of their work both laboratory and field testing procedures have been developed. In addition, research personnel at the Transportation Test Center, Facility for Accelerated Service Testing (FAST) track in Pueblo, Colorado, are currently collecting data on the performance of track substructures and hardware superstructures. Observations of in-service track are being made by Battelle Columbus Laboratory in Ohio.

Goldberg-Zoino & Associates, Inc. (GZA), is a geotechnical engineering consulting firm that specializes in evaluating the engineering performance of soil and rock materials and the design of building foundations, pavements, earth dams, and related earth structures. As part of the current research program sponsored by the FRA and conducted by TSC, GZA was contracted to perform a comprehensive review of current railroad substructure practices and technologies and the related engineering practices in the fields of soil and rock mechanics, geology, highway and airfield pavement design and evaluation, and associated geotechnical engineering fields. In this first volume of the Ballast and Subgrade Requirements Study, we will identify the most suitable technology and design criteria related to earth materials that could be directly applied to railroad track substructure design, maintenance, rehabilitation, stabilization, and performance evaluations. This report has been directed toward primarily practicing railroad engineers who regularly work with earth materials in track engineering applications. We have assumed that the reader has a familiarity with some of the basic principles of soil mechanics. Review of an elementary soil engineering textbook, such as Soil Mechanics in Engineering

Practice by Terzaghi and Peck (1967), may help prepare the reader who has little prior soil mechanics experience. We have also provided information for the railroad engineering researcher by describing the variety of procedures and practices encountered in the review. We have provided herein suggested practices for using earth materials in railroad track substructure and for stabilizing subgrade soils, based on our comprehensive review of available research and practices and our own expertise. In developing these recommendations, we have concentrated on those aspects of earth material evaluation, performance, and stabilization that could be of direct and immediate benefit to the railroad industry for analyzing candidate earth materials for use in track construction, for evaluating in-service earth materials for deficiencies and/or potential placement, and for stabilization of deficient materials. We have de-emphasized the use of engineering tests that require special equipment and operator training--such as cyclic triaxial tests--for evaluating earth materials because we do not believe that such tests are of immediate practical use to the railway industry except in special cases.

These earth material and stabilization practices are of limited application because they are based on earth material sciences and related engineering disciplines that are in themselves inexact. Our reasoning is as follows:

a. Even if elaborate exploration and materials testing programs are conducted, most engineering solutions are based upon testing relatively small earth material samples from isolated locations.

b. Subsurface conditions are heterogeneous, varying greatly from site to site and even within the confines of one site; this fact influences not only the in-situ subgrade materials but also the borrow sources for ballast and subballast. Design of stabilization measures must consider the variability of the substructure, although within some limit of variability.

c. Earth materials' stress-strain relations are nonlinear and depend upon many factors including time, environment, stress history, and the state of in-situ and applied stresses.

d. The idealized theoretical models such as the beam-on-elastic-foundation formulations or more complex computer programs such as GEOTRACK, ILLITRACK, ARTS, and PSA that are proposed for railroad track engineering analyses, only approximate the complex boundary conditions and stress-strain relations of the in-situ earth materials.

e. The empirical relationships representing the engineering performance of earth materials such as the hyperbolic stress-strain relation or the Mohr-Coulomb failure criterion are approximate.

f. The parameters' values such as resilient moduli and shear strengths derived from a group of engineering property tests usually involve a considerable degree of scatter.

In studying the suggested practices, these six factors must always be kept in mind.

In this report, we have provided uniform procedures for classifying and categorizing earth materials. Recommendations are presented to forecast potential use and performance of earth materials in track and to evaluate materials in the substructure of in-service track. Each element of the substructure is discussed separately, with the similarities among the elements pointed out. Section 2 is devoted to classifying, testing, and using natural soils as railroad track subgrades. Section 3 similarly addresses the classification and performance of crushed rock and rock-like ballast materials. In Section 4, the bridge between these two materials is made by reviewing earth materials used as subballast. Section 5 contains a summary of stabilization practices to repair or upgrade track subgrades. Section 6 contains a summary of findings and conclusions.

Appendix A contains a compendium of correlations between engineering performance parameters and index parameters for subgrade soils. Appendix B contains a collection of criteria used by various state transportation agencies in carrying out geotechnical investigations. Appendix B is intended to provide a guide in the planning of exploration programs for railroad track engineering. Appendix C contains some detailed recommendations for procedures to be used in completing those index property tests recommended in Section 3 for characterizing ballast material. Appendix D is an annotated bibliography of published case histories of subgrade stabilization programs for railroad tracks and related structures.

2. SUBGRADE SOILS FOR RAILROAD TRACK

In this report, subgrade soils are considered to be natural earth materials lying under the ballast and subballast. Placement and engineering properties of high fills beneath the subballast and stability of deep cuts are not covered.

Subgrade materials are expected to perform the following functions:

a. Support the track structure, ballast, and subballast

b. Accommodate the stresses due to train loads with acceptably small vertical and horizontal deformations

c. Maintain a stable position over time that is unaffected by such environmental factors as freezing temperatures, moisture changes, erosion, and infiltration of soil particles

d. Provide a suitable working base for construction of the subballast and ballast.

Knowledge of the type of subgrade soils that lie under a railroad route and their engineering properties are necessary to:

a. Determine whether or not subgrade soils will satisfactorily perform the previously mentioned functions

b. Design subballast and ballast sections that are compatible with subgrade soils and that will accommodate any deficiencies in the subgrade

c. Select and design, if appropriate, suitable subgrade stabilization measures for new track or to improve performance of in-service track

d. Select the route, compatible with geotechnical and other requirements, with the most favorable subsurface conditions.

2.1 SUBSURFACE INVESTIGATIONS

Successful design of railroad track substructure requires a program of field exploration and laboratory investigation, from which the type, extent, and engineering properties of the subgrade soils can be determined. Details of procedures and equipment for soils exploration are provided by Hvorslev(1), the U.S. Bureau of Reclamation (2), and the American Society of Civil Engineers (ASCE) (3) and serve as a basis for much of the following discussion.

---

(1) M. J. Hvorslev, Subsurface Exploration and Sampling of Soils for Civil Engineering Purposes, Report of the Committee on Sampling and Testing, Soil Mechanics and Foundations Division, ASCE, 1948.
(2) U.S. Bureau of Reclamation, Earth Manual, 1974.
(3) C. H. Dowding, ed., Site Characterization and Exploration, ASCE, 1979.

Reconnaissance Phase

The reconnaissance phase investigation consists of a "review of information available from published sources and previous investigations, supplemented by site reconnaissance," as suggested by the American Railway Engineering Association (AREA) Manual for Railway Engineering, 1978. Information available may include observations made during site visits, aerial photographs, geologic maps and reports, and records of past construction, including previous explorations and foundation design data. Principal sources of information include Federal agencies and departments, specifically the U.S. Geologic Survey, U.S. Coast and Geodetic Survey, U.S. Department of Agriculture, Soil Conservation Service, and state and municipal highway, building, and public works departments.

Definition of geology, with identification of surface soil deposits is the principal objective of this phase. The reconnaissance phase should include "walking the line" of the proposed locations, with field mapping and notes on:

a. Topography
b. Ledge outcrops
c. Type of vegetation
d. Type of trees; i.e., conifers or deciduous
e. Surface drainage
f. Physical features such as evidence of slope instability, subsidence, subsurface collapse, etc.

Exploration Phase

It is recommended that this investigation be accomplished in two stages. During the preliminary stage, the objective is to provide base data on principal geologic formations identified during the reconnaissance phase. This can be accomplished by relatively few widely spaced explorations.

A more detailed phase of exploration may be undertaken after having clarified the overall geologic framework and having identified specific strata to be studied in detail. The detailed phase (also called the exploration or specification phase) provides the engineer with specifics on the nature and extent of subsurface soil and water conditions along a route and samples for classification, index tests, and engineering property tests. Principal exploration techniques are machine-excavated test pits and borings. Geophysical explorations (seismic or electrical resistivity) may also be applicable because large areas can be explored rapidly and economically.

There are great differences of opinion regarding spacing and depth of explorations, as exemplified by the following:

a. AREA Manual (1978)--Recommends, for fill foundations, "Subsoil conditions should be explored to a depth at least equal to the width of the proposed fill or to competent material."

b. The American Association of State Highway and Transportation Officials--In its Standard T86-74, AASHTO recommends, "The depth of exploratory borings or test pits for roadbeds, airports, or vehicle parking areas should be at least 5 feet (1.5m) below the proposed subgrade elevation. Special circumstances may increase this depth. Borings for structures or embankments should extend below the level of significant influence of the proposed load, as determined by a subsurface stress analysis."

c. U.S. Navy Design Manual - Soil Mechanics, Foundations and Earth Structures (DM-7)--Contains "Requirements for Boring Layout" and "Requirements for Boring Depths," which are reproduced as Tables 2-1 and 2-2 of this report.

d. U.S. Navy Civil Engineering Design Manual (DM-5)--Recommends, for soil and subgrade investigations for roadways, "Maximum spacings along centerlines of proposed roads (of) 300 feet. In areas where soil profiles are not uniform...spacings of 100 feet or less may be required." Minimum exploration depths of 6 feet are recommended for cut and shallow fill sections. Depth of explorations for high fill sections are determined by settlement and stability considerations.

e. U.S. Bureau of Reclamation Earth Manual--Recommends, for line structures, maximum hole spacings of "about one-mile intervals for feasibility investigations and about 2,000-foot intervals for specification stage investigations. For major structures, holes at 100-foot intervals are often necessary."

Recognizing the disparate opinions and the possible confusion generated by the above-cited sources, the following guidelines are recommended:

Cuts--In cut areas, the principal concerns are:

a. Water table location

b. Type of soil to be excavated

c. Stability of cut slopes

d. Volumetric soil expansion (swell)

e. Frost heave potential

f. Nature of soils at cut subgrade

g. Construction problems inherent in working equipment within the excavation.

Guideline recommendations for exploration in cuts areas are as follows:

a. For cuts terminating above the water table, provide boring or test pit to 5 feet (1.5m) below proposed subgrade at average 500-foot (150m) spacing.

b. For cuts terminating below the water table, the depth of boring below subgrade elevation should be at least equal to 1.5 times the depth that the subgrade is below the water table. The reason is related to seepage into excavation from underlying previous strata, especially where potentially artesian conditions exist. Average distance between boring should be 500 feet (150m).

c. In areas of potential slope stability, provide additional explorations to whatever depth and lateral extent is necessary to define conditions behind the slope and below the toe.

d. Borings must penetrate weak strata, especially weak cohesive soils.

TABLE 2-1. REQUIREMENTS FOR BORING LAYOUT

Areas for investigation	Boring layout
New site of wide extent	Space preliminary borings so that area between any four borings includes approximately 10% of total area. In detailed exploration, add borings to establish geological sections at the most useful orientations.
Development of site on soft compressible strata.	Space borings 100 to 200 ft at possible building locations. Add intermediate borings when building sites are determined.
Large structure with separate closely spaced footings.	Space borings approximately 50 ft in both directions, including borings at possible exterior foundation walls, at machinery or elevator pits, and to establish geologic sections at the most useful orientations.
Low-load warehouse building of large area.	Minimum of four borings at corners plus intermediate borings at interior foundations sufficient to define subsoil profile.
Isolated rigid foundation, 2,500 to 10,000 sq ft in area.	Minimum of three borings around perimeter. Add interior borings depending on initial results.
Isolated rigid foundation, less than 2,500 sq ft in area.	Minimum of two dry sample borings at opposite corners. Add more for erratic conditions.
Major waterfront structures, such as dry docks.	If definite site is established, space borings generally not farther than 100 ft adding intermediate borings at critical locations, such as deep pumpwell, gate seat, tunnel, or culverts.
Long bulkhead or wharf wall	Preliminary borings on line of wall at 400 ft spacing. Add intermediate borings to decrease spacing to 100 or 50 ft. Place certain intermediate borings inboard and outboard of wall line to determine materials in scour zone at toe and in active wedge behind wall.
Slope stability, deep cuts, high embankments.	Provide three to five borings on line in the critical direction to establish geological section for analysis. Number of geological sections depends on extent of stability problem. For an active slide, place at least one boring upslope of sliding area.
Dams and water retention structures . .	Space preliminary borings approximately 200 ft over foundation area. Decrease spacing on centerline to 100 ft by intermediate borings. Include borings at location of cutoff and critical spots in abutment.
Highways and airfields	See NAVFAC DM-5 and NAVFAC DM-21 for general requirements for highways and airfields. For slope stability, deep cuts, and high embankments, see layout recommended above.

Reproduced from Soil Mechanics, Foundations, and Earth Structures by U.S. NAVFAC, p 7-2-11. Year of publication - 1971.

TABLE 2-2. REQUIREMENTS FOR BORING DEPTHS

Areas for investigation	Boring depth
Large structure with separate closely spaced footings.	Extend to depth where increase in vertical stress for combined foundations is less than 10% of effective overburden stress. Generally all borings should extend no less than 30 ft below lowest part of foundation unless rock is encountered at shallower depth.
Isolated rigid foundations	Extend to depth where vertical stress decreases to 10% of bearing pressure. Generally all borings should extend no less than 30 ft below lowest part of foundation unless rock is encountered at shallower depth.
Long bulkhead or wharf wall	Extend to depth below dredge line between 3/4 and 1½ times unbalanced height of wall. Where stratification indicates possible deep stability problem, selected borings should reach top of hard stratum.
Slope stability	Extend to an elevation below active or potential failure surface and into hard stratum, or to a depth for which failure is unlikely because of geometry of cross section.
Deep cuts	Extend to depth between 3/4 and 1 times base width of narrow cuts. Where cut is above ground water in stable materials, depth of 4 to 8 ft below base may suffice. Where base is below ground water, determine extent of pervious strata below base.
High embankments	Extend to depth between ½ and 1-1/4 times horizontal length of side slope in relatively homogeneous foundation. Where deep or irregular soft strata are encountered, borings should reach hard materials.
Dams and water retention structures . .	Extend to depth of ½ base width of earth dams or 1 to 1½ times height of small concrete dams in relatively homogeneous foundations. Borings may terminate after penetration of 10 to 20 ft in hard and impervious stratum if continuity of this stratum is known from reconnaissance.
Highways and airfields	Extend auger borings to 6 ft below top of pavement in cuts, 6 ft below existing ground in shallow fills. For high embankments or deep cuts, follow criteria given above.
Airfields	Extend auger borings to 10 ft below top of pavement in cuts or 10 ft below existing ground in shallow fills.

Reproduced from Soil Mechanics, Foundations, and Earth Structures by U.S. NAVFAC, p 7-2-12. Year of publication - 1971.

Fills--Principal concerns regarding subgrade below fills are as follows:

a. Settlement due to consolidation of compressible strata

b. Displacement of subgrade by shear failure of weak strata

c. Soft to medium clays are typically the most troublesome

d. Granular soils usually perform satisfactorily under fill.

Guideline recommendations for exploration in fill areas are as follows:

a. For low fills (less than about 20 feet), space borings about 1,000 feet (300m) apart. Recommended depth is at least equal to width of proposed fill or to competent material. Where fill is underlain by unstable soils, such as peat or soft clay, space borings no more than 500 feet (150m) apart. Depth should be to competent material.

b. For high fills (more than about 20 feet), space borings about 500 feet (150m) apart. Depth should be at least equal to width of fill or to competent material.

c. Where high fills are constructed over soils deposited by or in water (fluvial, lacustrine, glaciofluvial, etc.), at least half of borings should fully penetrate such deposits, but to depth not more than twice the fill width.

d. In areas of potential embankment instability and/or excessive settlement, provide additional borings to whatever depth and at locations necessary to define conditions. Borings must penetrate weak strata, especially weak cohesive soils.

2.2 SUBGRADE SOIL PERFORMANCE CHARACTERISTICS

Performance characteristics describe the aspects of soil behavior that directly relate to the ability of the subgrade soil to perform its intended functions. Soil performance characteristics are divided into four classes:

a. Mechanical--Related to the ability of soil to support the track structure, ballast, and subballast, and accommodate superincumbent loads--single, repeated, and dynamic--with small deformation.

b. Environmental--Related to the resistance of soil to changes from temperature, water, or other nonmechanical factors.

c. Permeability--Related to the passage of liquid (i.e., water) through the subgrade soils and infiltration or penetration of soil particles.

d. Construction--Related to the sensitivity of the soil to construction traffic and the workability of the soil as it is moved or altered during construction.

Mechanical Characteristics

The mechanical characteristics of soil relate to stress-strain behavior, as typically described by shear strength, volume change under quasi-static load, and deformability under transient load. The following factors represent the associated geotechnical issues:

Shear Strength

Shear strength relates to stability of cut slopes and stability of embankments constructed over weak soils. Typical practical situations are:

a. Cut slopes in cohesive soils, silt, or clay

b. Cut slopes below the water table in silt and/or fine sandy soils

c. Fills over cohesive and organic soils.

Quasi-static Volume Change

Consolidation--Soil porosity decreases, thus soil becomes denser under quasi-static applied stress. The volume decreases of soil porosity appear as surface settlement. Cohesive soils have time lag associated with settlement due to the slow rate of water flow from the soil. Typical subgrades subject to consolidation are:

a. Clays, plastic silts and organics

b. Highly micaceous residual soils (saprolite).

Sand, gravel, and most inorganic silt usually have negligible quasi-static compressibility.

Collapse--A special class of soils exhibits a relatively low compressibility when loaded dry but will suddenly collapse when it becomes saturated. The soil type that is most notorious for this behavior is loess, consisting mainly of wind-laid particles of silt and fine sand with a small amount of clay that binds the particles together. Loose, cemented sands and some residual soils derived from granite may also collapse. In its natural state, true loess has a characteristic open structure formed by the remnants of small vertical root holes that give it a low density and make it moderately pervious in the vertical direction. Upon wetting, the partially saturated loess may compress from 0.5 percent to 5.0 percent. Loess deposits are found in extensive portions of the plains areas of the northwest quadrant of the United States and the Mississippi Valley of the central United States.

The usual way to treat potentially collapsing soils is to saturate them prior to final grading. First fill is placed and then water is applied to the soil. It is necessary to wet the soil throughout the partially saturated zone. Vertical coarse sand drains have been used successfully to permeate the soil and produce the accompanying settlement more rapidly. Sometimes underground explosions are used to accelerate the process. Final grading and construction follows the precompression process.

Swell--This is the opposite of consolidation in that the soil porosity increases due to a new source of moisture and/or when the overburden stress is decreased as would occur in cuts. Highly plastic, very stiff, or hard clays existing above the permanent groundwater table are the most troublesome soils. Examples of environmental changes that provide moisture to the partially saturated soil are:

a. Covering an area prevents evaporation, thus moisture remains in ground and is taken up in soil by capillarity

b. Grading changes

c. Leakage through pavement

d. Cut drainage ditches

e. Percolation through railroad track ballast

f. Removal of vegetation.

Deformability Under Transient Load

In contrast to strength and compressibility, which may involve both near-surface and deep soils below fills, transient load deformability is controlled almost entirely by near-surface soils. The principal phenomena are:

a. Accumulation of strains and change in stiffness due to repeated loadings

b. Stresses and strains within the ballast and subballast due to displacement of the subgrade

c. Compression and rebound from passage of vehicles.

In the simplest sense, one engineering application of transient load deformability would be selection of subgrade modulus for analysis of the track-ballast-subgrade system, including complex analytical studies requiring computer modeling.

Fundamental Principles of Mechanical Performance

The strength and deformability characteristics of soils are governed by two soil mechanics fundamentals:

Effective Stress--The total normal stress (σ), acting at a point within a soil mass, is carried by the pore fluid in the voids and by the mineral skeleton. If the soil voids are saturated, the stress in the void phase is the porewater pressure (u). The average stress carried by the soil skeleton is the intergranular or effective stress ($\bar{\sigma}$), and is equal to the total stress minus the pore pressure, as shown in the equation, $\bar{\sigma} = \sigma - u$.

If the soil is dry, the pore pressure will be zero, and the effective stress will equal the total stress. If the soil is partially saturated, pore air and porewater pressures will differ and the effective stress will be difficult to determine. Most importantly, effective stress, rather than total stress, controls the deformation and shear strength behavior of soils.

Mohr-Coulomb Failure--This concept predicates the existence of a linear envelope (shown in Figure 2-1) that approximates the shear strength behavior of soils. If the Mohr circle for a given state of effective stress lies below the envelope, the soil strains will be stable. If the Mohr circle is tangent to the envelope, then the shear strength of the soil will be fully mobilized and the given state of stress will correspond to the peak of the soil's stress-strain curve. Large strains will develop if there is an attempt to further increase the shear stress. The point of tangency between the circle and envelope represents the stresses at failure on the failure plane that will develop within the soil mass.

The Mohr-Coulomb failure concept is described by the following equation:

$$\tau_{ff} = c + \bar{\sigma}_{ff} \tan \phi$$

where:

τ_{ff} = shear stress acting on failure plane at failure

$\bar{\sigma}_{ff}$ = effective normal stress acting on failure plane at failure

c = cohesion intercept of failure envelope

ϕ = slope of failure envelope or friction angle

One must have a good grasp of effective stress and failure theory to develop a basic understanding of soil behavior. A thorough examination of the subject is beyond the scope of this report. Thus, the reader is referred to standard soil mechanics tests, as for example by Terzaghi and Peck (1) and Lambe and Whitman (2).

---

(1) K. Terzaghi and R. Peck, Soil Mechanics in Engineering Practice, Second Edition, John Wiley & Sons, New York, 1967, 729 pp.
(2) T. W. Lambe and R. V. Whitman, Soil Mechanics, John Wiley & Sons, New York, 1969, 553 pp.

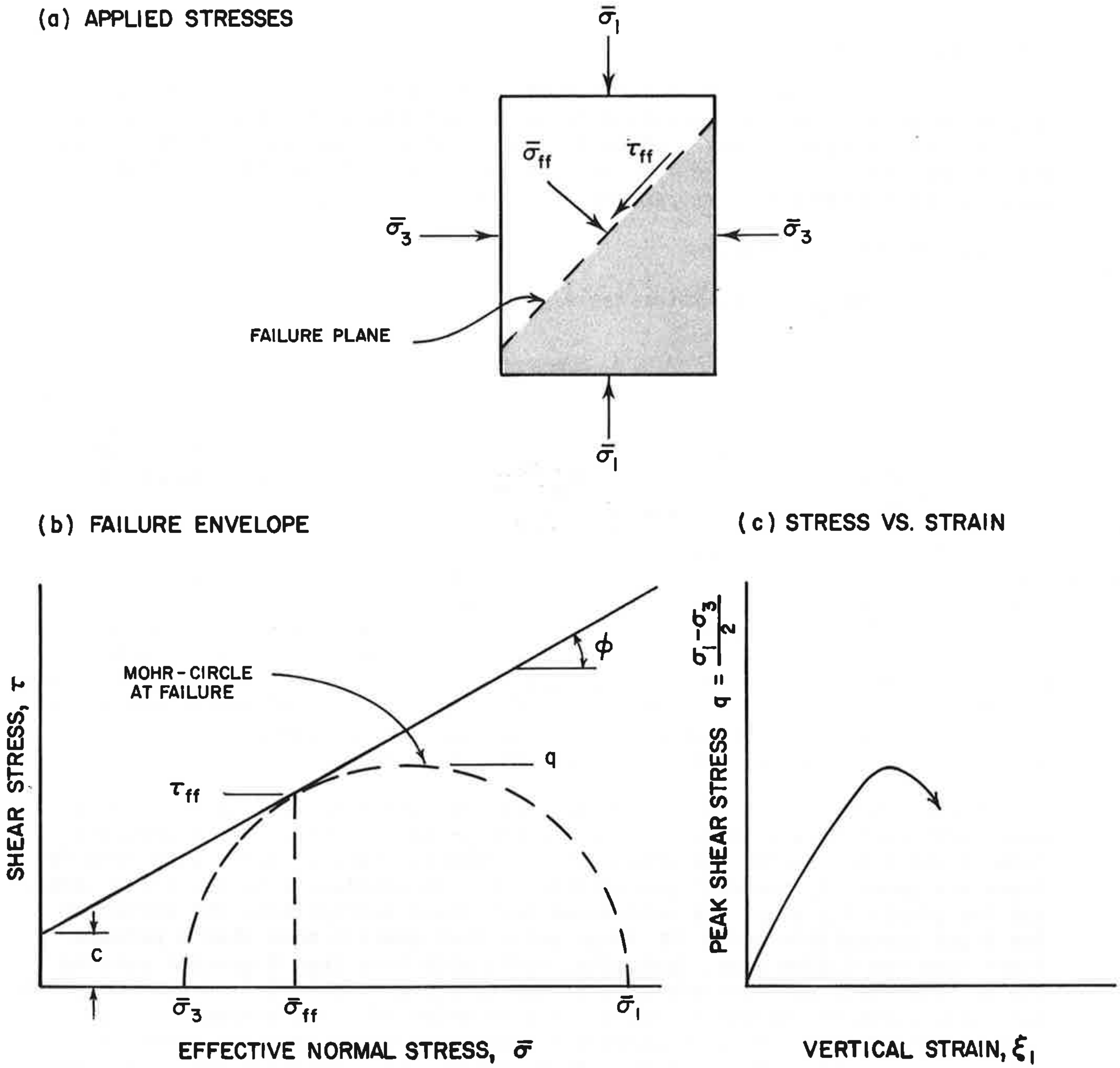


FIGURE 2-1. MOHR-COULOMB FAILURE CRITERION FOR SOILS

Environmental Characteristics

Shallow subgrade soils are subject to alteration due to environmental factors, particularly frost action and moisture.

Frost Action

Moisture is drawn into soil via surface infiltration and by capillary migration from below. In so-called frost susceptible soils, the ice segregates in distinct horizontal lenses above the water table. The result is frost heave during the freeze period and loss of subgrade support during the frost-melt period. Four conditions are required for ice segregation:

a. Freezing temperature

b. Freezing above the water table

c. A supply of water

d. A frost-susceptible soil.

The depth of frost penetration into subgrade soils is highly dependent upon the thickness of subballast and ballast, surface vegetation and snow cover, and the use of salt on the roadbed.

Ice segregation is a capillary-related process occurring above the water table involving moisture migration to the ice lens. Freezing of saturated soils, while possibly involving some expansion, does not lead to ice lens segregation. As a practical matter, unless in arid areas, water is available from groundwater or from surface infiltration. Mitigation measures, such as surface or subsurface drainage, will help but are not necessarily cure-alls. One condition where subsurface drainage is absolutely of critical importance is in cuts in sloping topography where underground seepage may concentrate in a particular zone, freeze, and lead to extraordinary heaving.

Frost-susceptible soils are those that are sufficiently fine-grained to have large capillary rise yet are sufficiently pervious to allow an adequate flow of water to nourish the growth of ice lenses. Granular soils with very few fines are generally nonfrost-susceptible, whereas nonplastic silts, silty sands, and low plasticity clays are considered most frost susceptible. One criterion for frost-susceptible soils is those soils that contain more than 3 percent finer than the 0.02mm size. Typically, soils with less than 8 percent passing the No. 200 sieve will satisfy this 0.02mm criterion. Table 2-3, prepared by the U.S. Army Corps of Engineers, rates soils in order of frost susceptibility. Class F1 is the least frost susceptible (least heave and greatest strength during thaw). Class F4 is the most frost susceptible (greatest heave and least strength during thaw).

TABLE 2-3. CORPS OF ENGINEERS' FROST DESIGN SOIL CLASSIFICATIONS

Frost Group		Kind of Soil	Percentage Finer than 0.02mm by Weight	Typical Soil Types Under Unified Soil Classification System
F1		Gravelly soils	3 to 10	GW, GP, GW-GM, GP-GM
F2	(a)	Gravelly soils	10 to 20	GM, GW-GM, GP-GM
	(b)	Sands	3 to 15	SW, SP, SM, SW-SM, SP-SM
F3	(a)	Gravelly soils	Over 20	GM, GC
	(b)	Sands, except very fine silty sands	Over 15	SM, SC
	(c)	Clays PI > 12	--	CL, CH
F4	(a)	All silts	--	ML, MH
	(b)	Very fine silty sands	Over 15	SM
	(c)	Clays, PI < 12	--	CL, CL-ML
	(d)	Varied clays and other fine-grained, banded sediments	--	CL and ML; CL, ML, and SM; CL, CH, and ML; CL, CH, ML, and SM

(1) From K.A. Linell, et al., "Corps of Engineers' Pavement Design in Areas of Seasonal Frost," Highway Research Record, No. 33, 1963, p. 79.

Moisture

Subgrade soils may become softened in the presence of water if they are compressible and are subjected to repeated heavy loads due to railroad traffic. This will result in soft spots, fouled ballast, mud pumping, ballast pockets, settlement of track, and continual maintenance requirements. The way that soft spots develop in track is similar to the development of pumping beneath rigid pavements. As paraphrased from page 284 of the AREA *Manual*, ". . . when track is laid in compressible subgrade soils, traffic loads will cause voids to form under the ties, due to accumulated plastic deformations in the subgrade soils, after the more elastic track structure rebounds. Water collects within these voids and softens the surrounding soil. With additional traffic loading, this softened material squeezes either to between the ties or laterally, to the track shoulder. Soft plastic clays are most susceptible to softening and disturbance in the presence of water."

Some fine-grained soils may experience large shrinkage when drying and swelling when absorbing water. This may be due to seasonal moisture changes or due to hydrologic conditions that are altered by railroad construction. Culverts and other locations where saturation followed by drying may occur are of particular concern. Possible design or remedial measures include crowning and sealing of the subgrade surface, chemical stabilization of shallow subgrade, or flooding prior to construction.

Permeability Characteristics

The coefficient of soil permeability is a measure of the velocity of fluid seeping through soil under a given hydraulic gradient. In granular materials, it is most fundamentally dependent upon soil grain size distribution and density. In silt-clay soils, permeability decreases with increased clay content--or putting it another way, with plasticity.

The coefficient of permeability (k) is defined as the discharge velocity (v) through a unit area under a unit hydraulic gradient, i.e.--

$$v = ki$$

where:

v = discharge velocity
i = hydraulic gradient (head loss per unit length of seepage path)
k = coefficient of permeability

Table A-8.1 in Appendix A presents ranges of typical permeability values for various types of soils.

Perhaps the single most important factor governing permeability of gravel or sand-gravel mixtures is the fines content. As noted in Table A-8.2 in Appendix A, the permeability of such excellent drainage materials as clean sand-gravel can be decreased by more than a factor of 1,000 if the fines content is around 15 percent. Clearly, the migration of fines (via surface infiltration or pumping from below) can destroy the usefulness of subballast or ballast materials as a drainage material.

Gradation and soil density also govern permeability of sands and mixtures of sand-gravels as discussed below.

Gradation - Because the ease with which water flows through soil depends upon the smallest constrictions through which it passes, finer sand particles of a coarse-grained soil will have the greatest influence on permeability. For example, Hazen, in a "Discussion of 'Dams on Sand Foundations'" in 1911, proposed that the permeability of sands is proportional to the square of the D particle size, i.e.--

$$k = 100\ (D_{10})^2$$

where:

k = permeability coefficient in centimeters per second
D_{10} = the sieve size through which 10 percent by weight of the soil will pass, in centimeters

The Hazen formula is still used in practice to approximate permeabilities of sandy soils.

Further requirements, based upon overall gradation range, as well as using a certain particle size as key indices have been proposed by Burmister. Further elaboration is beyond the scope of this report. Additional correlations are provided in Appendix A.

Density - Looser soils having more void volume will be correspondingly more permeable than denser soils. An increase in void ratio, or decrease in density, will increase soil permeability because the size of the flow channels will increase. Based upon theoretical and experimental work summarized by Lambe and Whitman (1), variations of permeability with void ratio that have been proposed are as follows:

---

(1) T. W. Lambe and R. V. Whitman, Soil Mechanics, John Wiley & Sons, New York, 1969, pp. 289-291.

$$k \propto e^2/(1+e) \text{ ,or } k \propto e^3/(1+e) \text{ ,or } k \propto e^2 \text{ ,or } k \propto \exp(e)$$

where:

k = permeability
e = void ratio
exp = denotes exponentiation

Construction Characteristics

Subgrade soils, particularly in cuts, are inevitably subjected to construction operations that may affect performance of not only the subgrade but of subballast and ballast as well. Soils that are sensitive to disturbance, particularly in the presence of water, will be softened if heavy construction vehicles are allowed to pass directly on them. If left in place, this softened material will result in increased settlement and decreased stability of the track section. Accordingly, limitations on construction procedures may be required to limit construction disturbance. Common techniques are switching to lightweight equipment or to drag lines or backhoe operating from a shelf level above the final subgrade. Those soils that are particularly sensitive to construction disturbance are very soft clay, silt, and silty fine sands.

2.3 TESTS FOR EVALUATING SUBGRADE SOILS

Descriptions and evaluations of engineering property tests, index property tests, and field tests that can be used to evaluate subgrade soil performance characteristics are presented in this section.

Laboratory Engineering Property Tests

There are a variety of laboratory tests which, when performed on undisturbed or reconstituted soil samples, measure the subgrade soil performance characteristics described in the previous section. Test results are strongly influenced by such factors as sample disturbance, sample preparation procedures, and system testing errors (e.g., non-uniform boundary stress conditions). Therefore, results must always be evaluated and corrected if necessary, and applied to in-situ conditions using sound engineering judgment. The following paragraphs contain brief descriptions of five of these engineering property tests most applicable to railroad substructure engineering.

Consolidation (Oedometer) Test

Purpose - This test is used to determine soil compressibility for settlement analyses such as to predict long-term settlements and settlement rate beneath railroad embankments or heave due to cuts in cohesive soils. Tests are almost exclusively carried out on undisturbed samples of silt or clay. Tests on compacted specimens are performed less frequently, perhaps only for extremely high embankments.

Specifics of Test - The basic features of the consolidation test apparatus are shown in Figure 2-2. In this test, vertical loads are applied--usually by means of dead weights--to a cylindrical specimen while lateral strain is prevented by a confining ring. Thus, only vertical strains occur. The sample diameter is usually about 60mm although samples as large as 100mm and more are used on occasion. Drainage from the sample occurs near the porous stones at the top and bottom surfaces of the sample.

Typically, the vertical load is applied in 12-hour or 24-hour increments to a maximum of 16 tons per square foot pressure, then rebounded to zero load, again in increments. Time readings of compression during each increment provide a means to predict the rate of consolidation in situ.

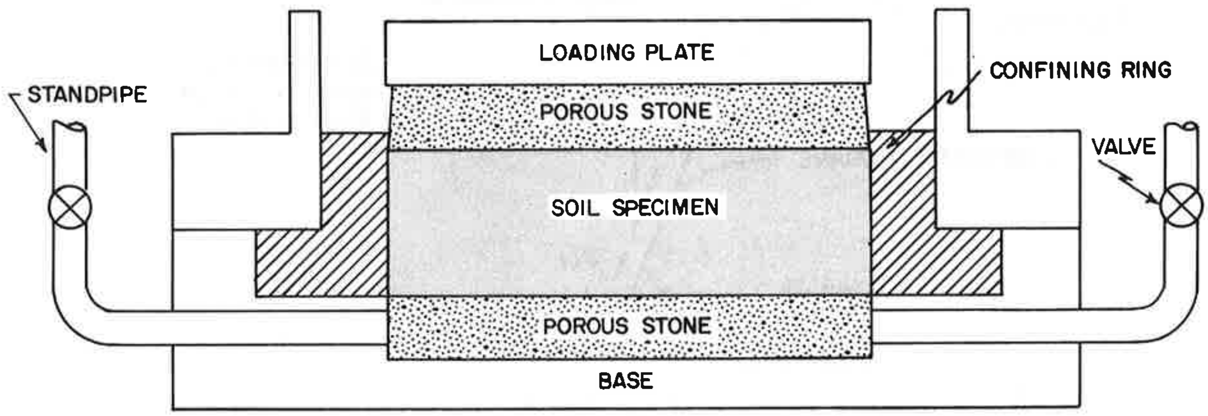
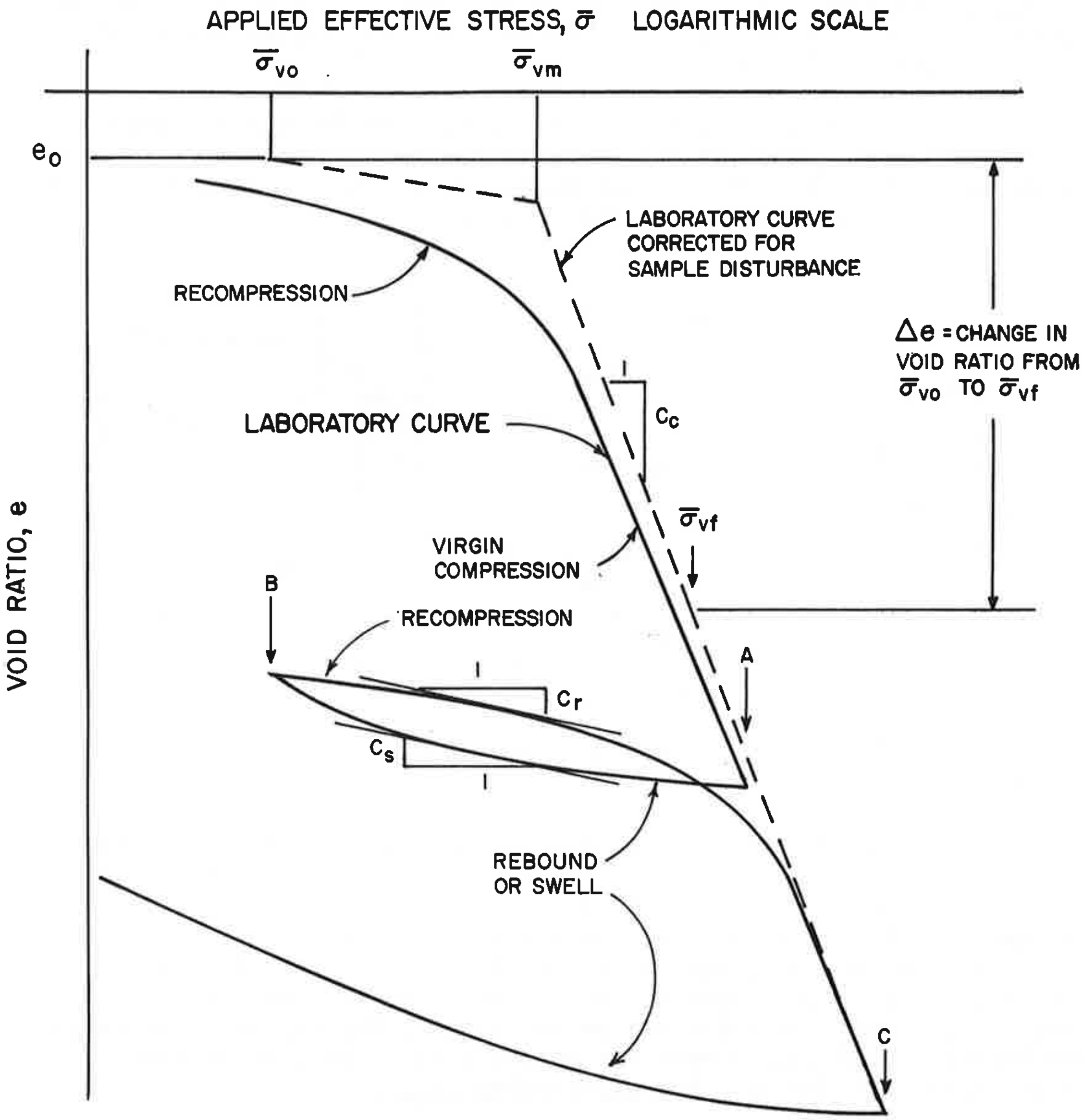


FIGURE 2-2. FIXED RING CONSOLIDOMETER

Test Results - Consolidation test results are presented in terms of the void ratio attained after compression under an applied effective stress versus the logarithm of the applied effective stress. Figure 2-3 illustrates the results of a consolidation test in which the sample was first loaded to effective stress A, unloaded to effective stress B, and then reloaded to effective stress C. The in-situ void ratio (e_o) and vertical overburden effective stress ($\bar{\sigma}_{vo}$) are also indicated on the figure. The stress at which the loading curve becomes steep (and linear for many clays) is considered the maximum past pressure that the soil has experienced ($\bar{\sigma}_{vm}$).

Clays are categorized as normally consolidated or overconsolidated, depending on the relative values of $\bar{\sigma}_{vo}$ and $\bar{\sigma}_{vm}$. The maximum past pressure for normally consolidated soils is equal to the in-situ overburden effective stress. The in-situ, load-settlement relations will be governed by the steep virgin compression curve, and in-situ settlements will be relatively large.



$\bar{\sigma}_{vo}$ = IN-SITU EFFECTIVE OVERBURDEN STRESS
$\bar{\sigma}_{vm}$ = MAXIMUM PAST PRESSURE
$\bar{\sigma}_{vf}$ = IN-SITU FINAL EFFECTIVE STRESS
e_o = IN-SITU VOID RATIO
C_c, C_s, C_r = AVERAGE LOGARITHMIC SLOPES, $\Delta e / \Delta \log \bar{\sigma}$, ON VIRGIN COMPRESSION, SWELL, AND RECOMPRESSION CURVES, RESPECTIVELY

FIGURE 2-3. TYPICAL LABORATORY CONSOLIDATION CURVE

The maximum past pressure for overconsolidated soils is larger than the in-situ overburden stress because of historical factors, such as glaciation, man-made excavations, or natural erosion. The degree of overconsolidation is quantified by means of the overconsolidation ratio, OCR, defined as follows:

$$OCR = \bar{\sigma}_{vm} / \bar{\sigma}_{vo}$$

For overconsolidated clays, the in situ load settlement relations will be governed by the flat recompression portions of the consolidation curve, provided the final stress (initial stress plus applied stress) does not exceed the maximum past pressure. In-situ settlements will be relatively small, typically about 10 percent to 15 percent of compression that might occur in the virgin range of normally consolidated soils.

Discussion - The advantages of using this test is that it's simple to perform and the one-dimensional state of strain corresponds to many field-loading situations. In addition, many experiences documented in the literature provide ample guidance for present-day applications.

The major difficulty of the test is the side friction that occurs between the sample and the confining ring that alters the uniform state of stress within the sample. This effect can be minimized most effectively by limiting the thickness to diameter ratio of the sample to approximately 1:3 or 1:4. In addition, specially lubricated confining rings or floating ring consolidometers are occasionally used to minimize the effects of side friction. However, their use may introduce setup difficulties and disturbance effects that mitigate their effectiveness.

The consolidation test is most frequently applied to settlement problems on soft, cohesive soils, and measures compressibility under one-dimensional loading and strain conditions. Though rarely used in typical geotechnical applications involving granular soils, the test may offer some promise in railroad applications.

The consolidation test apparatus is also used to study swelling potential for active soils. In the unrestrained swell test, the suspect soil is loaded in the consolidometer to a small pressure such as 5 to 10 kPa (about 1 psi). The drainage stones are then flooded. If the soil has a swelling potential, the soil will absorb water and the accompanying vertical expansion strain of the sample may be measured. A measured strain less than 1.5 percent is considered low. A strain greater than 5 percent is high, above 25 percent is very high.

In the swelling-pressure test, consolidation test samples are loaded dry to a pressure two or more times the existing overburden. Water is then introduced into the porous end stones and the resulting swelling strains may be measured. The confining pressure is then successively reduced and the succeeding swell is observed. From these data, a relation between confining pressure and swelling strain may be developed. This relation can be used to evaluate the confining pressure required to eliminate swell or to predict the swell amount anticipated in the field.

Collapsing soils may be identified and evaluated by performing a double oedometer test similar to that used for swelling soils. Two tests are carried out in a similar manner on nearly identical samples. However, in one test, no water is introduced into the porous stones, whereas, in the companion test, water is introduced to the sample after a small seating load is applied. The difference in the strain measured at each pressure in the two tests is indicative of soil structure collapse.

There are several difficulties in testing swelling and collapsing soils. It is difficult to obtain "identical samples" for the double oedometer tests. The results are extraordinarily sensitive to disturbance effects that may be caused by sampling in the field or preparing the laboratory test specimen. Finally, the results are dependent upon securing a sample in the laboratory that is at the same water content as the soil in the field at the time of construction. This is a function of both sampling procedures and natural variation in soil moisture content that occurs over time in-situ.

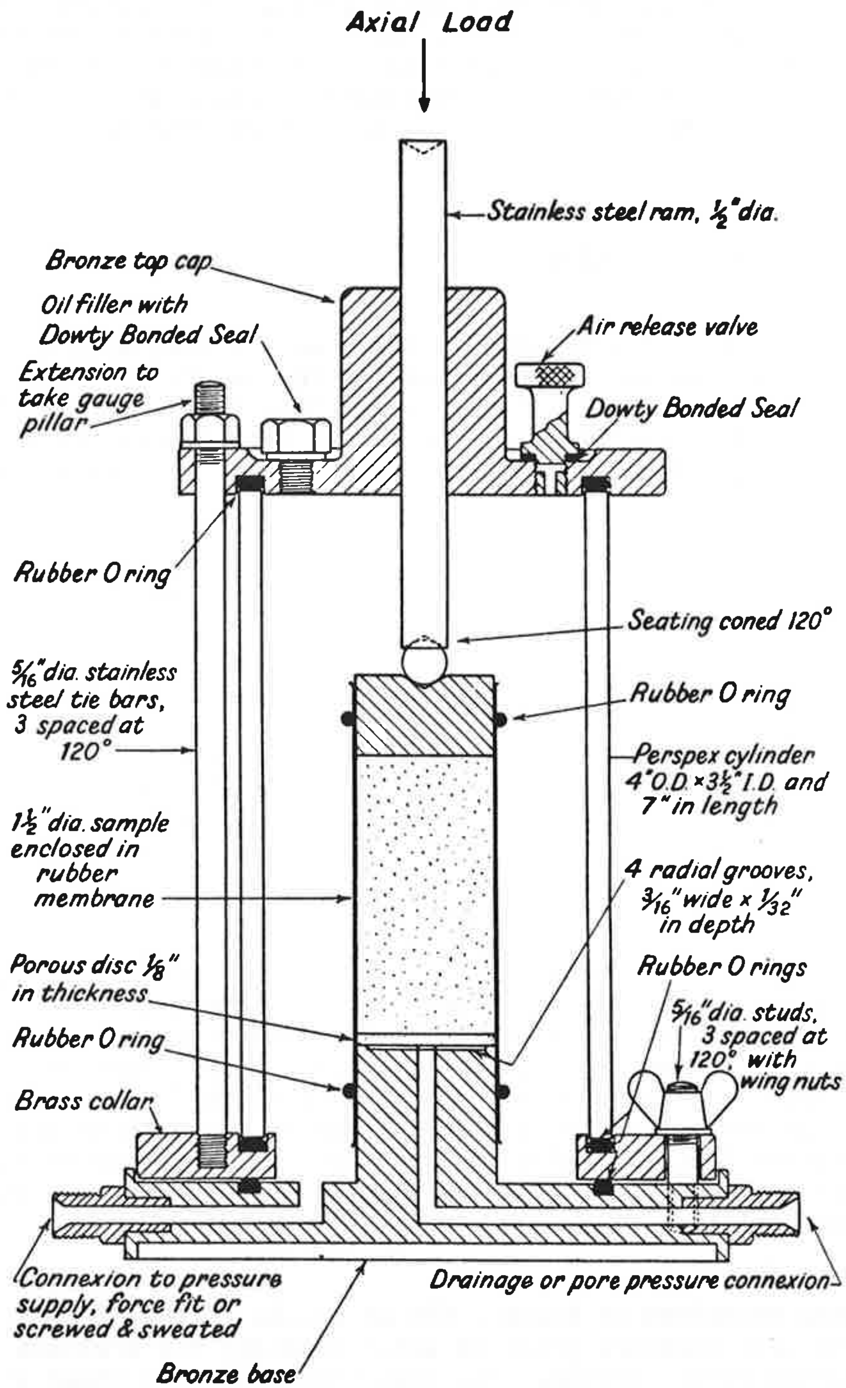
Triaxial Test

Purpose - The most common purpose is to determine soil shear strength properties as may be used in analyses of slope stability for cuts and embankment construction. Both undrained strength (S_u) and drained strength Mohr-Coulomb parameters (c, ϕ) may be determined in the triaxial test. Additionally, the deformation modulus, relating axial strain to stress, can be determined for a wide range of drainage conditions and of applied lateral and vertical stress conditions. Tests are typically performed on undisturbed samples of silt or clay. They can also be carried out on reconstituted samples of granular soils.

Specifics of Test - The apparatus used in the triaxial test is depicted in Figure 2-4. It consists of a chamber, which is pressurized by either gas or fluid (usually water), and a piston for applying vertical load to the sample. The cylindrical triaxial sample is sealed within a rubber membrane. Common sample diameters for testing soils are 35mm to 75mm, although some specialized test samples may be as large as 1m in diameter.

In the most common triaxial test, the chamber is first pressurized, subjecting the sample to an equal all-around confining stress, and then a vertical piston load is applied until the sample fails. As the load is applied, either at a constant rate of strain or in controlled stress increments, the sample will compress vertically and bulge laterally. Thus, this test is designated as a compression-loading test.

Axial compression, applied load, and lateral pressures are measured for the duration of the test by mechanical gauges or electronic measuring and recording devices. Lines through the pedestal of the triaxial cell provide access to the pore fluid of the soil sample. Thus, drainage conditions can be controlled, volume changes can be measured for drained conditions, and pore pressures measured for undrained conditions.



Reproduced from The Triaxial Test, p. 34, by A. W. Bishop and D. J. Henkel by permission of St. Martins Press. Year of first publication: 1962.

FIGURE 2-4. TRIAXIAL TEST CELL

Test Results - The simplest form of presentation is a plot of applied total stress versus axial strain. When pore water pressure is measured, the test result presentation would also include plots of pore pressure and effective stress against axial strain. Where drainage is allowed during the test, as described below, volume change is conventionally plotted against axial strain. Results are commonly used in analyses of stability of embankments and of cuts in cohesive soils.

Types of Triaxial Tests

Unconfined Compression--The test is performed on clay with zero confining pressure. Because of the soil's cohesiveness, the sample is not surrounded or confined by a membrane within a triaxial cell, thus the term "unconfined." Common practice assumes that one-half the unconfined compressive strength is equal to the undrained shear strength of clay. The test is rapid and inexpensive.

Because of difficulty in obtaining test results that can be consistently repeated (due to naturally occuring geologic variations and anomalies), the unconfined compression test is not always a reliable means of determining undrained shear strength.

Unconsolidated-Undrained or Quick - (UU or Q)--The sample is placed within a triaxial cell, surrounded by a membrane, cell pressure applied, but no drainage is allowed--thus the term "unconsolidated." The only difference between the unconfined compression and the unconsolidated-undrained test is the all-around confining pressure in the latter.

Consolidated-Drained or Slow - (CD or S)--As the term implies, the specimen is first allowed to consolidate under the cell pressure and to drain during axial load application. It is essential to load the specimen slowly to avoid buildup of excess pore pressure within the sample. The resulting drained shear strength ($S_d = \tau_{ff}$) is evaluated using the Mohr-Coulomb failure envelope. This drainage condition corresponds to field problems in which there are no excess pore-water pressures (due to a slow loading rate, a long lapse of time since the end of construction, or rapid drainage) and only hydrostatic or a steady state of water pressure exists.

Consolidated-Undrained or Rapid - (CU or R)--As in the CD test, drainage occurs under the cell pressure prior to axial loading; but drainage is discontinued during axial loading. The resulting undrained shear strength ($S_u = \tau_{ff}$) can be related to the effective consolidation pressure at the start of the test or can be interpreted in terms of effective stress at failure for tests in which pore pressure measurements are made during shear.

Discussion - The outstanding advantages of the triaxial test are the provisions for independent control of the principal stresses in vertical and horizontal directions and control of drainage conditions. This permits application of stress and drainage conditions that correspond to many field situations. The major disadvantage of the triaxial test is the inability to independently control the principal stresses in the two horizontal directions. Thus, some in-situ stress-strain conditions--such as plane strain loading beneath a long embankment strip load-- can be modeled only approximately by the triaxial apparatus. Many true triaxial test devices that are capable of varying the stresses independently in three directions have been developed during the past 15 years, however, there remain major equipment problems that preclude their widespread use.

Experimental difficulties associated with the triaxial test include boundary and end effects (largely minimized by use of specimen height to diameter ratios of at least 2 to 3) and sample disturbance effects. Strength data derived from triaxial tests are applicable to stability analyses of cut slopes, fills, and foundation loads. The deformation moduli are applicable to calculation of subgrade strains. Moreover, the test can be carried out in a manner that simulates field conditions of static or cyclical loading. (1,2)

Direct and Simple Shear Test

Purpose - The specific purpose of these tests is to determine strength parameters under simulated field conditions of failure along a horizontal plane. Triaxial data may be used also, but because the triaxial test does not precisely model field conditions, it may not be applicable to more sophisticated analyses. In addition, the direct and simple shear tests provide for anistropic consolidation of specimens in the apparatus prior to shear which is more like the field condition than isotropic consolidation that is normally carried out in a triaxial cell.

Specifics of Test - The basic features of the direct shear apparatus are shown in Figure 2-5. The apparatus consists of a box (split across the middle) and systems for applying vertical normal load and horizontal shear load. The direct shear sample is circular or square in plane with sample height of about 25 mm and cross-sectional area of 100 mm to 150 mm. Larger shear boxes are available for testing aggregates or rock joint surfaces.

---

(1) T.W. Lambe. "The Stress Path Method," Journal of the Soil Mechanics and Foundation Division, ASCE, Vol. 93, No. SM6, November 1967, pp. 300-331.
(2) T.W. Lambe and W.A. Marr, "Stress Path Method: Second Edition," Journal of the Geotechnical Division, ASCE, Vol. 105, No. GT6, June 1979, pp.727-738.

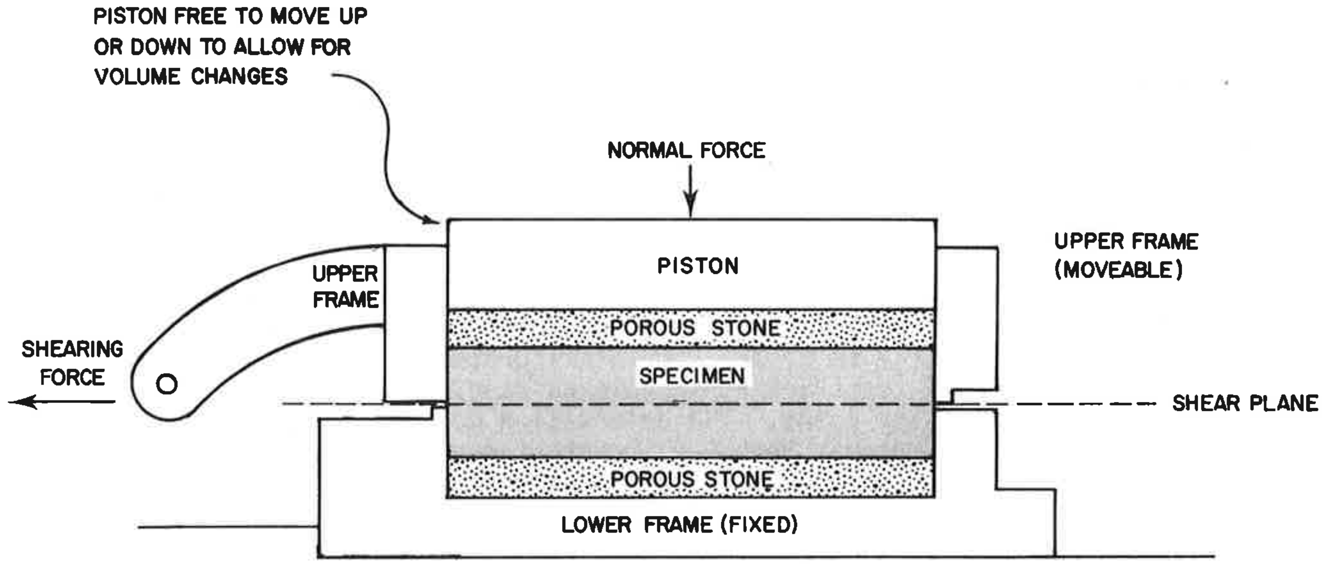


FIGURE 2-5. SCHEMATIC OF DIRECT SHEAR BOX

The direct shear test is performed by first applying vertical load to the soil and then allowing consolidation. The horizontal shear load is then applied, shearing and splitting the sample at mid-plane. Most direct shear equipment lacks any provision for drainage control, and the test is best performed slowly as a fully drained test. In 1962, O'Neill (1) described a modification of the test in which the normal load is varied during shear to maintain constant sample volume and, hence, to approximate undrained conditions.

Test Results - Results are usually presented as a plot of displacement (horizontal and vertical) versus shear stress. In drained tests, volume change during test is also reported.

---

(1) H.M. O'Neill, "Direct Shear Test for Effective Strength Parameters," Journal of the Soil Mechanics and Foundation Division, ASCE, Vol. 88, No. SM4, August 1962, pp. 107-137.

Discussion - The main disadvantages of the direct shear test include severely non-uniform stresses at the boundaries and within the sample, a resulting progressive failure mode within the sample, and an indeterminate stress system. Because of these difficulties, use of the test is limited particularly for determining general stress-strain properties of soils.

Bjerrum and Landva (1) and Roscoe (2) describe direct-simple shear devices that apply more uniform stresses and strains to the sample than does the direct shear apparatus. Although not without difficulties (e.g., boundary stress concentrations at corners or edges of samples, indeterminate stress system, expense, and availability of equipment), the simple shear test has the outstanding advantage of simulating the rotation of principal planes that occurs in many embankment loading problems. Foote and Ladd (3) also describe the application of simple shear test results to a variety of problems for undrained loading on soft soils.

In recent years, cyclic simple shear tests have been used to evaluate the behavior of soils during earthquake loadings. In this application, the cyclic simple shear test simulates the action of vertically propagating horizontal shear waves, which are an important component of earthquake motion.

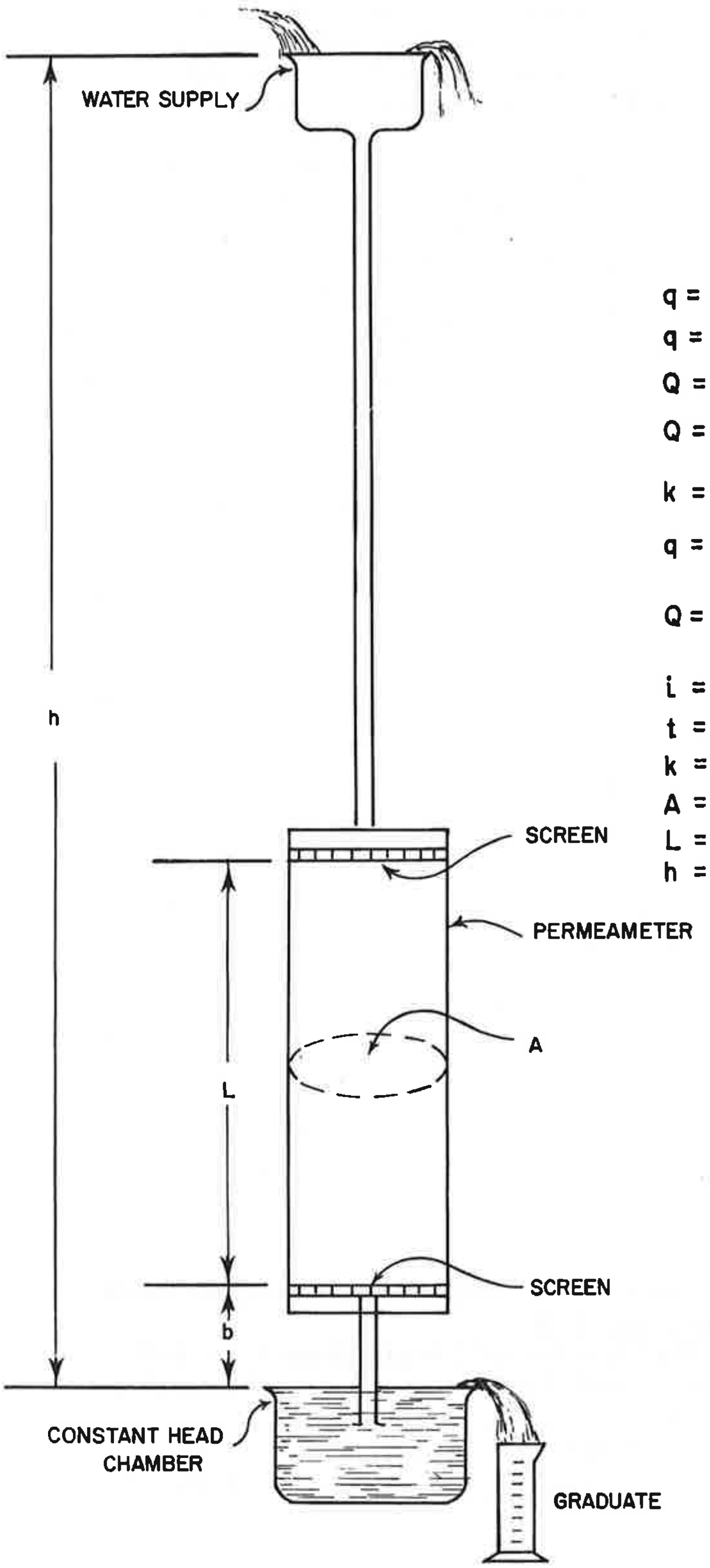
Permeability Test

Purpose - The permeability test determines the coefficient of permeability for analyses of seepage and groundwater flow. These results might be used to size drainage facilities for temporary or permanent lowering of the groundwater level.

Specifics of Test - Constant and falling head permeameters for direct laboratory measure of soil permeability are shown in Figures 2-6 and 2-7, respectively. For the constant head test, seepage rate under constant head conditions is measured and the permeability is then computed directly by Darcy's Law, as shown in Figure 2-6. For the falling head test, Darcy's Law is applied to the time rate of head loss for a computation of permeability by means of the equation shown in Figure 2-7. The falling head test is best used for low permeability soils in which seepage rates are too small to be precisely measured by a constant head test.

---

(1) L. Bjerrum and A. Landva, "Direct Simple Shear Tests on a Norwegian Quick Clay," Geotechnique, Vol. 16, No. 1, 1966, pp. 1-20.
(2) K.H. Roscoe, "An Apparatus for the Application of Simple Shears to Soil Samples," Proceedings of the Third International Conference on Soil Mechanics and Foundation Engineering, Switzerland, Vol. 1, 1953, pp. 186-191.
(3) C.C. Ladd and R. Foote, "New Design Procedure for Stability of Soft Clays," Journal of the Geotechnical Engineering Division, ASCE, Vol. 100, No. GT7, July 1974, pp. 763-371.



$$q = kiA$$
$$q = k \cdot \frac{h}{L} \cdot A$$
$$Q = q \cdot t$$
$$Q = k \cdot \frac{h}{L} \cdot A \cdot t$$
$$k = \frac{QL}{hAt}$$

q = QUANTITY OF SEEPAGE THROUGH SAMPLE PER UNIT AREA

Q = QUANTITY OF SEEPAGE THROUGH SAMPLE

i = HYDRAULIC GRADIENT, h/L

t = TIME

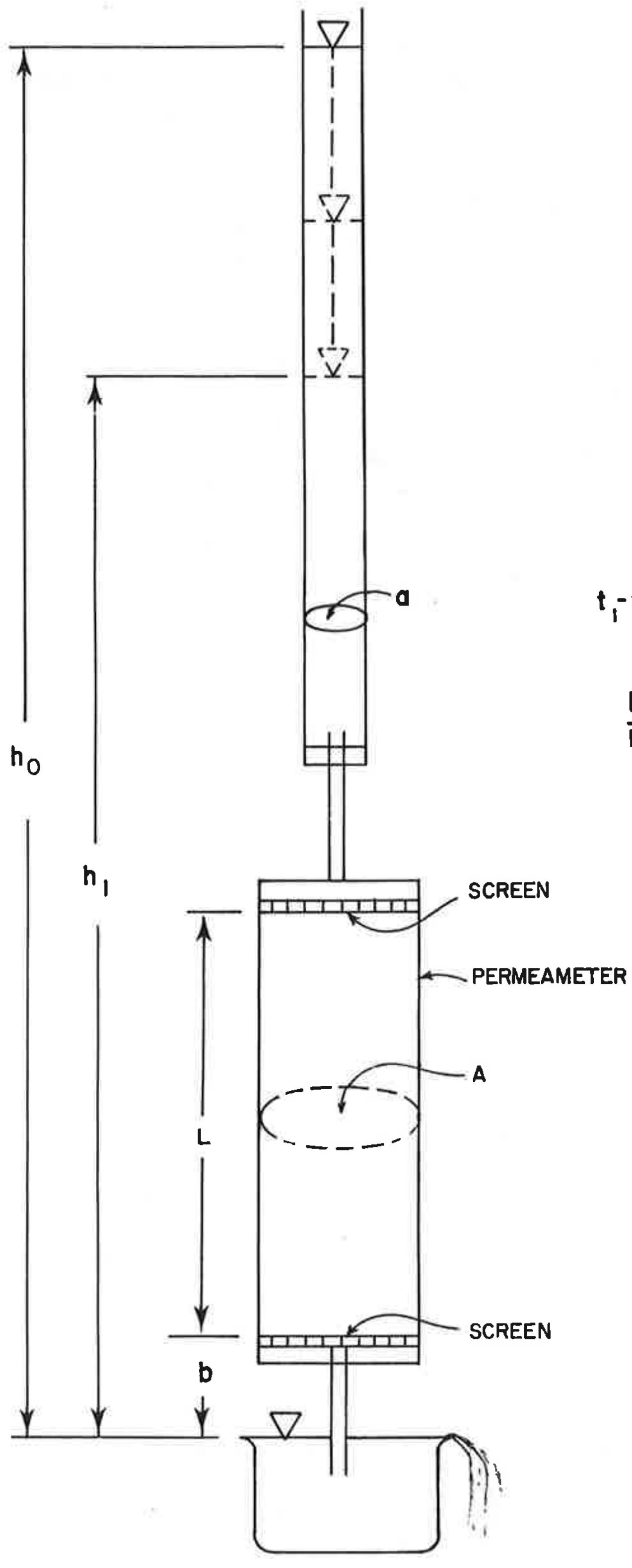
k = COEFFICIENT OF PERMEABILITY

A = AREA OF PERMEAMETER

L = LENGTH OF SAMPLE

h = TOTAL HEAD CHANGE ACROSS SAMPLE

FIGURE 2-6. SETUP FOR CONSTANT-HEAD PERMEABILITY TEST



$$k = \frac{aL}{A(t_1 - t_0)} \log_e \left(\frac{h_0}{h_1}\right)$$

a = AREA OF STANDPIPE

A = AREA OF PERMEAMETER

$t_1 - t_0$ = TIME FOR WATER LEVEL IN STANDPIPE TO FALL FROM h_0 TO h_1

$\frac{h_0}{h_1}$ = RATIO OF HEADS AT t_0 AND t_1

L = LENGTH OF SAMPLE

FIGURE 2-7. SETUP FOR VARIABLE-HEAD PERMEABILITY TEST

Soil permeability may also be determined in the triaxial test. A vertical seepage gradient is established with the use of a drainage top cap; the seepage rate can then be measured and permeability computed by using Darcy's Law.

Test Results - Computed permeability value is reported, giving applicable test conditions (head, gradients, time, etc.) upon which the computation is based. Computations are based on the equations shown in Figures 2-6 and 2-7.

Discussion - Laboratory permeability tests on granular soils provide a useful measure of the effects of changes in particle size, gradation, and void ratio on permeability. However, there are numerous difficulties with the experiments, including filter skin effects due to migration of fines and development of large flow channels along the boundaries of the sample. Moreover, laboratory tests on granular soils do not reflect in-situ stratification that, without exception, results in greater horizontal permeability than vertical permeability. Finally, most tests on granular soils are run on reconstituted samples, another factor that does not reflect stratification. Because of these difficulties, laboratory permeability tests are of limited practical value for assessing the absolute magnitude of in-situ permeability.

Dynamic Laboratory Tests

The two dynamic tests most applicable to railroad engineering are the resonant column and cyclic triaxial tests. The purposes of these tests are as follows:

a. To evaluate the plastic or nonrecoverable strain induced by cyclic loading

b. To evaluate cyclic or resilient moduli to provide material parameters for analyses of track system response.

The British Rail has developed standardized cyclic triaxial testing procedures that are used to develop track substructure designs on stiff clay soil subgrades (1). However, use of cyclic test procedures in North America has been limited to providing input parameters for analyses by various computer programs that model resilient displacement and permanent settlement of track (2).

---

(1) "Repeated Loading of Clay and Track Foundation Design," ORE-Report D71/RP12, October 1970, pp. 30-33.
(2) R.M. Knutson et al., "Materials Evaluation Study--Ballast and Foundation Materials Research Program," U.S. DOT, Federal Railroad Administration, Report No. FRA-OR&D-77-02, January 1977, pp. 217-265.

Resonant Column Test

The resonant column apparatus consists of a basic triaxial apparatus that is modified to permit high-frequency oscillation of a cylindrical soil sample by means of an electromagnetic device. The resonant frequencies of the sample are then measured, and the sample modulus is computed by means of simple wave propagation concepts. Equipment is available that will excite the sample torsionally for determination of soil shear modulus (G) or longitudinally for determination of Young's modulus (E). The test also provides a direct measure of hysteretic soil damping.

The major disadvantage of the resonant column test is sample disturbance effects, particularly for granular soils. In addition, present-day equipment is limited to small strains (less than 0.1 percent).

Cyclic Triaxial Test

The cyclic triaxial test has experienced considerable development and use in earthquake engineering applications since the mid-1960s when it was first applied to soil liquefaction problems. In this test, the cylindrical triaxial sample is subjected to cyclic vertical compression and extension loading, and the stress-strain characteristics are directly measured. Relatively simple and inexpensive pneumatic cyclic loaders are used primarily for evaluating liquefication behavior of saturated soils. More expensive and elaborate servo-hydraulic units provide a direct measure of the soil's stress-strain curve during cyclic loading. From this curve, soil modulus (E), and hysteretic damping are directly determined.

Although potentially an extremely useful test for railroad applications, the cyclic triaxial test has a number of limitations. Nonuniform boundary stresses at the end platens may redistribute water or grains within the soil sample. In addition, test results are extremely sensitive to sample disturbance and sample preparation procedures. Finally, measured cyclic triaxial behavior of soils greatly depends upon the applied cyclic stress system. For example, Lee and Seed (1) show that whether or not a 90-degree reorientation of the maximum principal stresses occurs during the loading cycle greatly affects the measured strains. Thus, research on the stress systems applied to the subgrade during train loading and development of appropriate cyclic testing procedures is necessary.

---

(1) K.L. Lee and H.B. Seed, "Dynamic Strength of Anisotropically Consolidated Sand," Journal of the Soil Mechanics and Foundation Division, ASCE, Vol. 93, No. SM5, Part 1, September 1967, pp. 169-190.

Laboratory Index Property Tests

Soil index properties can be correlated, at least approximately, with engineering properties. Thus, a fully integrated testing program should always include both index tests and engineering properties tests (a simple example is to always run a gradation test with a permeability test on granular soil). Other samples with similar gradation would be expected to have similar permeability--i.e., gradation is an index of permeability. Such correlations serve as a basis for feasibility studies and preliminary design even though the index tests provide only an approximate measure of material performance. In detailed engineering analyses and tests for final design, such index correlations with engineering properties provide means to anticipate and extrapolate actual measured soil behavior on a site-specific basis. A collection of correlations between index test parameters and engineering properties is provided in Appendix A of this report.

The following paragraphs describe the index property tests that are judged most suitable for evaluating railroad subgrade soils.

Gradation

Gradation or grain size distribution of coarse-grained soils (i.e., sands and gravels) can be determined by passing dry soil through a set of sieves that are stacked with progressively smaller openings from top to bottom, and by weighing the amount of soil retained on each sieve. Grain size distribution of fine-grained soils (e.g., silts and clays) is determined by hydrometer analysis that measures the critical velocity at which particles of varying size fall out of suspension (1). For mixed-grained soils that contain fine- and coarse-grained particles, both sieve and hydrometer analyses are performed.

The results of the grain size tests are generally plotted as a grain size distribution curve showing the percentage of the soil sample by weight that is smaller than a particle diameter size versus the logarithm of the equivalent particle size. Typical grain size curves are shown in Figure 2-8.

The location and shape of the distribution curve is important to note. For example, a steep vertical curve indicates a very uniform-sized soil (see curve A in Figure 2-8); a large horizontal portion of the grain size curve indicates a gap-graded material that is deficient in particle sizes within certain limits (see curve B, Figure 2-8); and a flat, gradually sloping curve indicates a very well-graded soil (see curve C, Figure 2-8). The grain size curve is often described by the following parameters:

D_n = Grain size (D) of which n percent of soil weight is finer than

-No.(M)sieve = Material finer than the openings of a standard sieve containing M openings per square inch

+No.(M)sieve = Material coarser than the openings of a standard sieve containing M openings per square inch

(1) T. W. Lambe, Soil Testing for Engineers, John Wiley & Sons, New York, 1951, pp. 29-42.

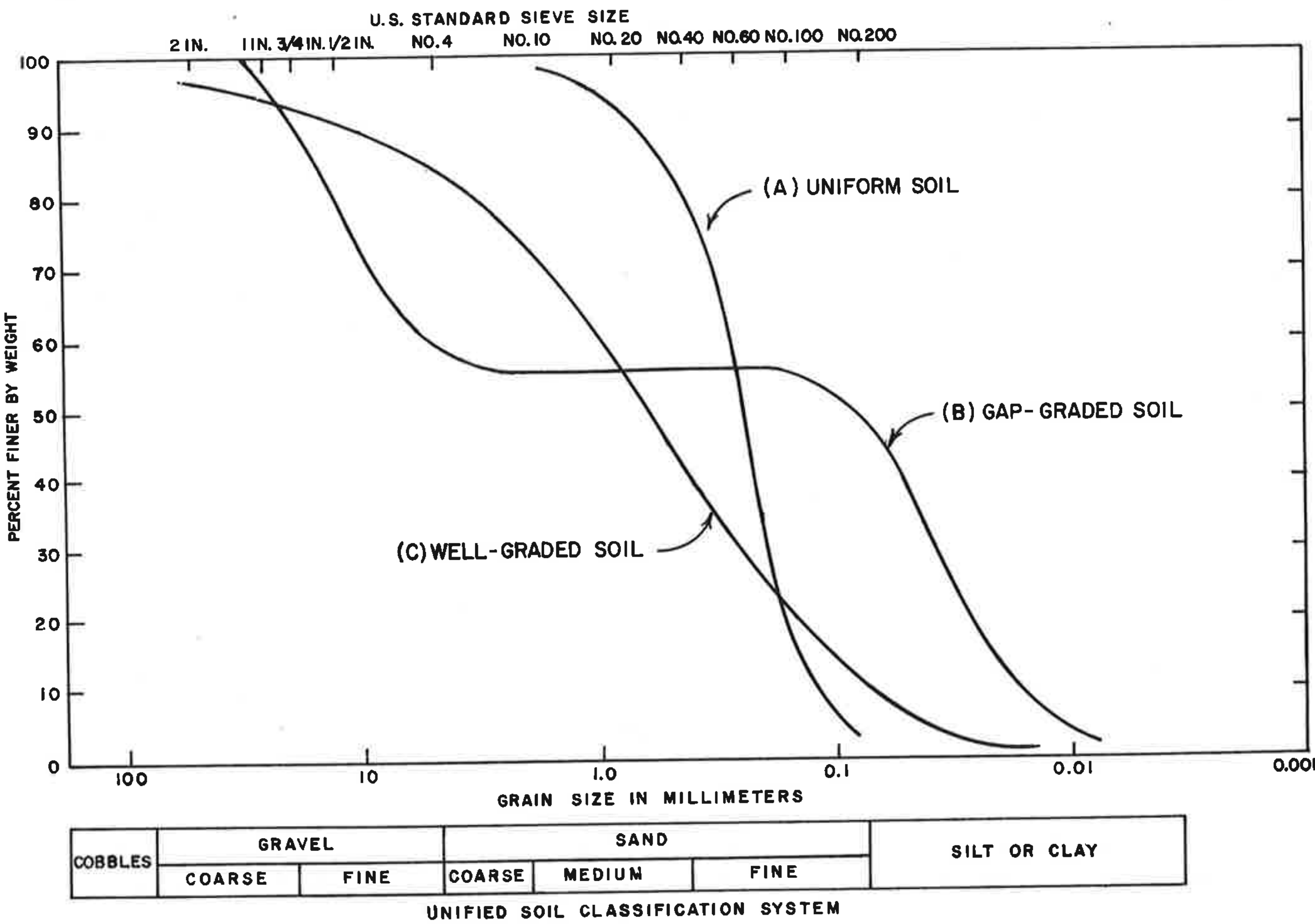


FIGURE 2-8. TYPICAL GRAIN SIZE CURVES

For example, the D_{10} size of coarse-grained soils is frequently used as an index of permeability, and the percentage of -No. 200 sieve material significantly influences freeze-thaw resistance, drainage potential, and sensitivity to disturbance of soils.

Coefficients describing the slope and shape of the grain size curve are:

Coefficient of Uniformity-- $C_u = D_{60}/D_{10}$

Coefficient of Curvature-- $C_c = (D_{30})^2/(D_{10} \times D_{60})$

These coefficients indicate whether a coarse-grained soil is well-graded (i.e., $1<C_c<3$ and $C_u>4\text{-}6$) or poorly graded (i.e., uniform or gap-graded).

Particle sizes determined by the sieve and hydrometer tests only are an index of actual sizes of particle grains. In the sieve analysis, sieve size is determined by the length of the sides of the square openings in the mesh. The size of the particles that pass through a sieve is dependent on the particle's shape since elongated or flat particles may pass through the sieve openings depending upon their elongation.

Phase Relationships

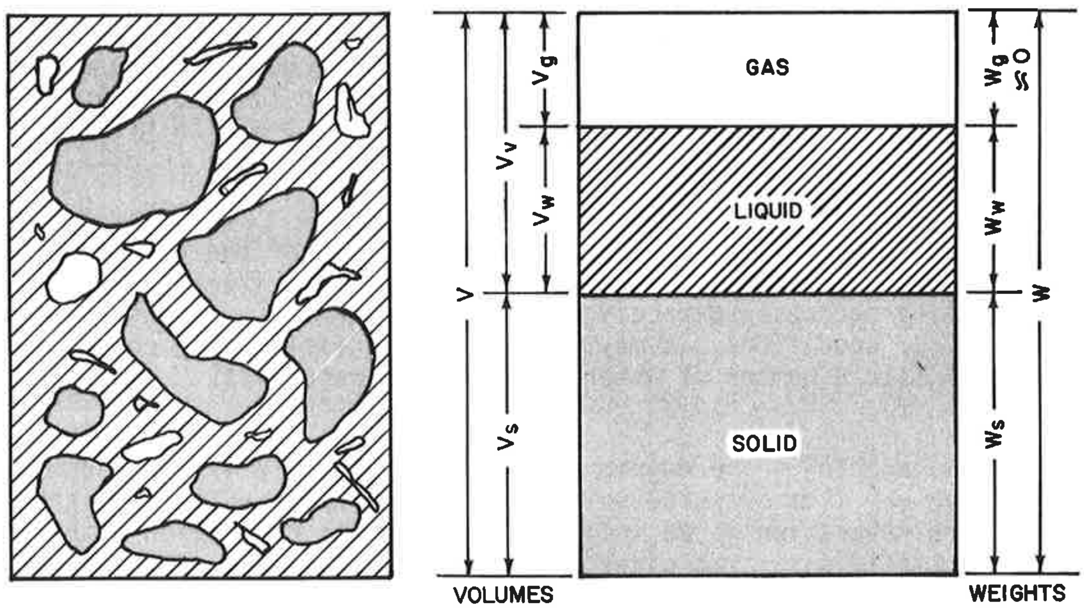
As shown in Figure 2-9, a multiphase soil system is usually represented by separating the solid, water, and air phases. Numerous phase-weight-volume relationships can be determined from simple measures of weights and volumes in the laboratory. Of these relationships, density, void ratio, and water content are the most important indices of soil performance. Degree of saturation and specific gravity are less important.

Relative Density - Relative density of granular soil can be computed using the following equation based only on void ratio (void volume divided by solids volume):

$$D_r = \{(e_{max}-e)/(e_{max}-e_{min})\} \times 100$$

Alternatively relative density can be computed by the following equation on the basis of dry density (dry unit weight):

$$D_r = \frac{\gamma_{d\,max}}{\gamma_d} \quad \frac{\gamma_d - \gamma_{d\,max}}{\gamma_{d\,max} - \gamma_{d\,min}} \quad \times 100$$



(a) NATURAL SOIL (b) IDEALIZED MODEL

DENSITY

TOTAL: $\gamma_t = \frac{W}{V}$

DRY: $\gamma_d = \frac{W_s}{V}$

SATURATED: $\gamma_{sat} = \frac{W}{V}$ WHEN $V_g = 0$

WATER: $\gamma_w = \frac{W_w}{V_w} = 62.4$ pcf $= 1$ g/cc

γ_{sub}: $\gamma_{sub} = \gamma_{sat} - \gamma_w$

VOID RATIO: $e = V_v/V_s$

WATER CONTENT: $w = W_w/W_s$

DEGREE OF SATURATION: $S_r = V_w/V_v$

SPECIFIC GRAVITY OF SOLIDS: $G_s = W_s/V_s/\gamma_w$

V = TOTAL SOIL VOLUME

V_s = VOLUME OF SOLIDS

V_v = VOLUME OF VOIDS

V_w = VOLUME OF WATER

V_g = VOLUME OF GAS

W = TOTAL SOIL WEIGHT

W_s = WEIGHT OF SOLIDS

W_w = WEIGHT OF WATER

W_g = WEIGHT OF GAS

FIGURE 2-9. PHASE RELATIONS FOR SOILS

where:

e_{min} = void ratio of soil in densest condition
e_{max} = void ratio of soil in loosest condition
e = in-situ void ratio
$\gamma_{d\ max}$ = dry density of soil in densest condition
$\gamma_{d\ min}$ = dry density of soil in loosest condition
γ_d = in-situ density

There are significant inherent testing problems, and determining relative density is highly unreliable. Nevertheless, relative density is frequently used because of the many correlations that have been developed for evaluating the performance of granular soils, including settlement of footings on sand, friction angle, and liquefaction potential. (See Section 2.4 and Appendix A.) These correlations should be used only when unavoidable, and with great caution and full appreciation of the inherent precision involved.

Water Content - Water content is an important index of the performance of fine-grained soils. With all other things being constant, strength decreases and compressibility increases directly with moisture content. With reasonably consistent geologic conditions, it may be possible to empirically correlate moisture content with a number of other engineering properties.

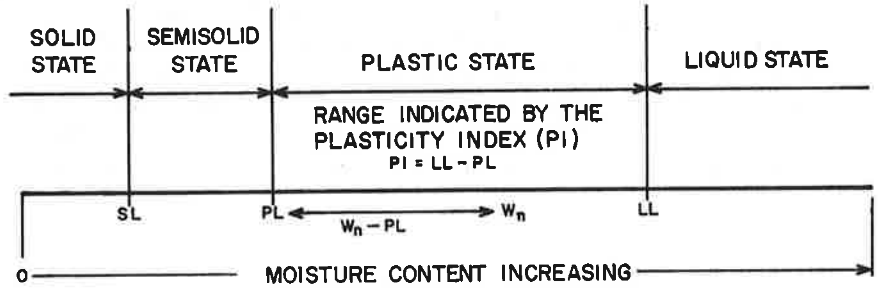
Degree of Saturation - The degree of saturation is determined by computing void and water volume from measured weights, total volume, and unit weights. One example where saturation is an important index is in connection with swell potential of partially saturated clays.

Specific Gravity of Solids - Specific gravity is the ratio of the density of the soil grains to the density of water at standard conditions. This is determined by measuring water displaced by a known weight of soil solids. Care must be taken to control water temperature and remove air entrapped in the water to obtain test results that can be consistently repeated. The range of specific gravity for most soils is small (i.e., 2.5 to 2.8). Thus, there's little reason to use it to measure soil performance except as an indicator of organics or of an extremely unusual soil type.

Plasticity

Plasticity, the ability to be molded, is exhibited by clay and some silts. Granular soils, except those with sufficient amount of clay, are nonplastic. As shown in Figure 2.10, a soil is friable when dry and with increasing water content will become plastic and finally liquid. The water contents at which the soil's states change are the Shrinkage Limit, Plastic Limit, and Liquid Limit.

Atterberg Limits are used to compute Plasticity and Liquidity indices (see Figure 2-10). They are important indices of soil behavior for most soil engineering problems involving fine-grained soils. For example, the liquidity index--a measure of the relationship between natural water content and the liquid and plastic limits--is an important indicator of stress history (e.g., degree of overconsolidation), shear strength, and compressibility of saturated cohesive soils.



PLASTICITY INDEX = LL - PL

LIQUIDITY INDEX = $\frac{W_n - PL}{LL - PL}$

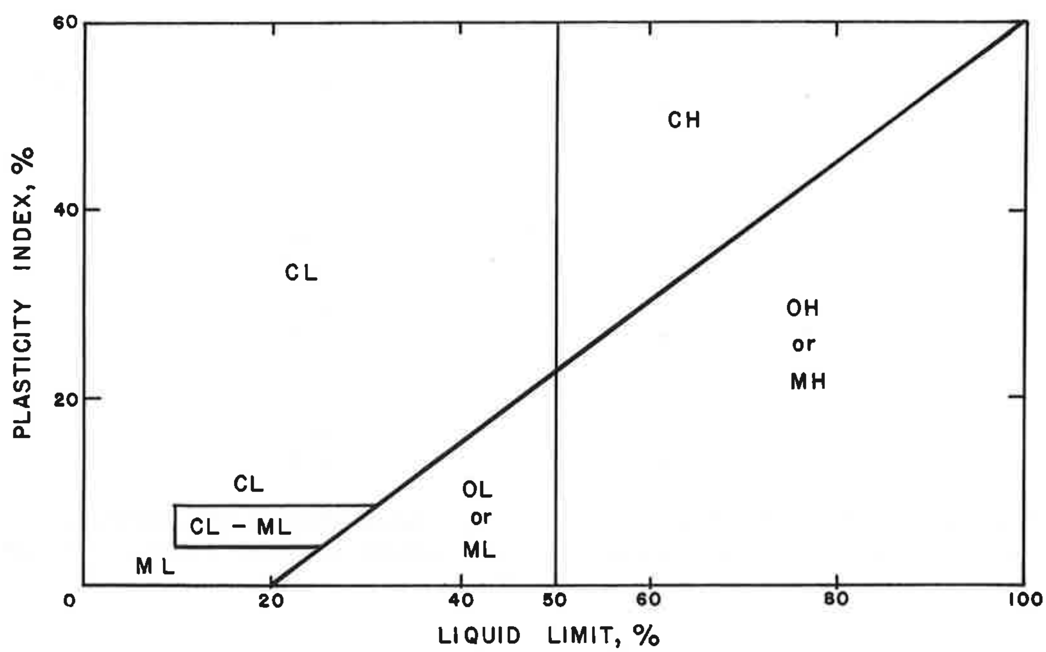
SL = SHRINKAGE LIMIT

PL = PLASTIC LIMIT

LL = LIQUID LIMIT

W_n = NATURAL WATER CONTENT

FIGURE 2-10. ATTERBERG LIMITS



After "Classification and Identification of Soils" by A. Casagrande, 1948.

FIGURE 2-11. PLASTICITY CHART FOR FINE-GRAINED SOILS

The plasticity chart, originally developed by Casagrande, plots plasticity index versus the liquid limit (see Figure 2-11). By plotting the Atterberg Limits of a soil on this chart, an engineer can determine general performance characteristics by correlation with known performance characteristics of soils that are plotted in the same zone on the chart. The Atterberg Limit tests are relatively simple and inexpensive to conduct, and their results can be repeated consistently.

Laboratory Bearing Resistance

Many of the tests shown in Table 2-4 measure the resistance of soil samples to vertical loads. The unconfined compression and California Bearing Ratio (CBR) Tests are believed to be the most applicable to railroad engineering problems. They are described in greater detail in the following paragraphs.

Unconfined Compression - As described previously in this section, the unconfined compression test is technically a triaxial test; however, because results are not always repeatable, it is perhaps more appropriately used as an index test. The unconfined compression test is conducted using a rapid application of vertical load to a cylindrical soil sample and measuring the relationship between vertical stress and vertical strain. The unconfined compression strength (q_u) is defined as the peak vertical stress on the resulting stress-strain curve, or the vertical stress at an arbitrary large strain (usually 12 percent or 20 percent) if the stress-strain curve does not peak.

The unconfined compression strength (q_u) is an index of in-situ shear strength, and serves as a basis for the verbal description of soil consistency that was developed by Terzaghi and Peck (1) in 1948:

Unconfined Strength (tons/sq.ft.)	Consistency
<0.25	Very soft
0.25 - 0.50	Soft
0.50 - 1.00	Medium
1.00 - 2.00	Stiff
2.00 - 4.00	Very Stiff
> 4.00	Hard

If it were possible to obtain a perfectly undisturbed, homogenous, isotropic sample, the undrained shear strength would be approximately equal to one-half the unconfined compressive strength.

(1) K. Terzaghi and R.B. Peck, Soil Mechanics in Engineering Practice, Second Edition, John Wiley & Sons, New York, 1967, p. 30.

TABLE 2-4. DATA FOR MAKING ESTIMATES OF PROBABLE VOLUME CHANGES FOR EXPANSIVE SOILS

Colloid content, as a percentage minus 0.01 mm	Plasticity index	Shrinkage limit	Probable expansion as a percentage of total volume change[a]	Degree of expansion
(a) Data from Index Tests[b]				
>28	>35	<11	>30	Very high
20-31	25-41	7-12	20-30	High
13-23	15-28	10-16	10-20	Medium
<15	<18	>15	<10	Low
Percentage passing No. 200 sieve	**Liquid limit, as a percentage**	**Standard penetration resistance, in blows per foot**	**Probable expansion as a percentage of total volume change**	**Degree of expansion**
(b) Laboratory and Field Data[c]				
>95	>60	>30	>10	Very high
60-95	40-60	20-30	3-10	High
30-60	30-40	10-20	1-5	Medium
<30	<30	<10	<1	Low

[a]The three soil properties are not to be used separately, but all three must be considered to arrive at the estimated degree of expansion from air dry to saturated conditions.

[b]Based on vertical loading of 1.0 psi.

[c]Based on loading of 1,000 psf.

Laboratory Vane Tests - Simplest type is a four-bladed, hand-held device that is pushed into the clay with blades penetrating about one-quarter inch, then rotated until shear failure occurs along the circle generated by the blades. The torque is calibrated to indicate a shear strength. A hand-held version of such a device is called the "Torvane." Another, somewhat more sophisticated laboratory vane apparatus is a motor-driven, four-bladed device. This vane penetrates into the sample; then the test is performed essentially the same way as the hand-held vane described above.

Sensitivity - The ratio of undisturbed strength to strength of completely remolded soil is defined as sensitivity (S_t). Using a vane test, one can continue to rotate a vane device after the peak is reached; the strength (torque) would fall off to a lesser, yet constant, value reflecting the residual shear strength after failure. A remolded unconfined test can be performed by squeezing the soil while it is contained in plastic or a rubber membrane to preserve moisture, placing it in a sample mold, and performing a second compression test.

The sensitivity of clay soils is frequently categorized as follows:

S_t	Description
2 - 4	Insensitive
4 - 8	Sensitive
8 - 16	Extra sensitive
> 16	Quick

California Bearing Ratio Test (CBR)-- The CBR test was developed by the California Highway Department in the late 1930's to design flexible (i.e., bituminous) pavements. The test consists of measuring the load required to cause a standardized piston, having an end area of 19.35 square centimeters (3 square inches) to penetrate the soil at 2.54, 5.08, 7.62, 10.16, and 12.70mm. The CBR is computed as the load causing a given penetration divided by standard penetration values for a high quality crushed stone material as follows:

Penetration (mm)	(in)	Standard Bearing Resistance Value (MPa)	(psi)
2.54	0.1	6.9	1000
5.08	0.2	10.3	1500
7.62	0.3	13.1	1900
10.16	0.4	15.9	2300
12.70	0.5	17.9	2600

CBR at a 2.54mm penetration is usually used for pavement design. When the CBR test is performed in a laboratory, a standard mold (178mm deep, 152mm diameter), plunger (penetration area of 19.35 sq. cm), and penetration rate (1.27mm/min.) must be used. Other features of the test such as soaking procedures and the use of surcharge weights have also been standardized. The test is most frequently performed on undisturbed, remolded, or compacted laboratory samples, although it can be performed in the field on undisturbed subgrade soils.

The CBR test provides an index measure of soil strength and the effects of softening due to soaking. As such, it has served as a basis for the design of flexible highway and airfield pavements. Yoder and Witczak (1) describe pavement design procedures developed by the U.S. Army Corps of Engineers, The Federal Aviation Administration, and The Asphalt Institute. Because of the similarity between highway and railroad engineering problems, the CBR test may be a reliable index test for the design of railroad ballast and subballast sections.

Organic Content

Organic matter in the form of partly decomposed vegetation significantly affects the performance characteristics of subgrade soils. Highly organic soils such as peats are easily identified by the color, odor, and the obvious presence of vegetable matter. An engineer usually doesn't need further index tests to evaluate the unsatisfactory performance characteristics of these highly organic soils.

The effect of the organic content on performance characteristics for many organic silts and clays may be more subtle, and therefore, an engineer may require index property measurements--such as the Atterberg Limits and unconfined compression strength--to evaluate performance characteristics. In addition, the organic content may be determined from the loss of weight upon ignition. The standardized ASTM test (D2974) uses an ignition temperature of 550 degrees C, which is presumed to be sufficient to ignite all but the mineral constituents. The ignition loss test is a particularly useful index measure for making a distinction between organic and inorganic silts.

Minerology

Minerologic composition will have a significant bearing on the performance characteristics of some soils. For example, a small mica content in a granular material may result in very high void ratios and high compressibility when subjected to load. Montmorillonite can cause a clay to be highly expansive and halloysite can result in it having very low unit weight. The cation exchange capacity of clay will greatly influence soil structure (i.e., flocculated versus dipersed) and hence, its performance.

Minerologic index tests that are frequently performed include cation exchange capacity and X-ray diffraction tests. In 1965, Black (2) described some methods for determining the cation exchange capacity of soils. All methods generally consist of saturating the cation exchange sites of the clay particles with a particular cation (e.g., Ba^{++}, Ca^{++}, Na^{+}) displacing this particular cation with another cation, and measuring the amount of displaced cation in the leachate.

---

(1) E.J. Yoder and M. W. Witczak, Principles of Pavement Design, Second Edition, John Wiley & Sons, New York, 1975.
(2) C.A. Black, Methods of Soil Analysis-Part 1, Serial No. 9, American Society of Agronomy, 1965.

Soil minerals may be determined by using X-ray diffraction analysis of thin sections. In this type of analysis, the diffraction pattern of an X-ray beam passing through thin sections is compared with the diffraction patterns for standardized reference samples. In 1970, Carrol provided a guide to X-ray identification of minerals, and Carver followed up in 1971 with details for preparing thin sections. The minerologic tests described in this section don't provide a quantitative measure of engineering performance; they simply indicate minerologic features that may affect engineering performance. As such, they serve as a guide in evaluating possible engineering problems and in selecting engineering performance tests.

Frost Susceptibility

Frost heave occurs when groundwater is drawn up from a moderate depth by capillary action into a subfreezing temperature zone near the surface. The process creates segregated ice lenses and heave of the surface during freezing, then subgrade weakening and settlement during thaw. Frost susceptibility includes consideration of both heave and weakening.

The soil must have an intermediate range of permeability and grain size for ice lenses to form. If the soil is very clean, it won't permit a sufficient capillary rise. It it's too clayey, soil permeability will be low, so that the rate of ice buildup won't be significant during the freezing season.

The U.S. Army Corp of Engineers (1) utilizes percent fines less than 0.02mm size as one criterion (i.e., index of frost susceptibility). Referring to Table 2-3 presented in Section 2-2, soils with greater than 3 percent finer than 0.02mm display some degree of frost susceptibility. Note that the more coarse-grained soils are least frost susceptible (F1 group) primarily because of improved drainage and shear strength features. The more fine-grained sands and silty sands are in the most frost susceptible group (F4), even though the percent finer than 0.02mm is the same as some of the gravelly soils. Thus percent finer than 0.02mm cannot be the sole criterion for frost susceptibility.

The frost susceptibility test procedure commonly employed was developed by the U.S. Army Corps of Engineers, Cold Regions Research and Engineering Laboratory (CRREL), and the Transport and Road Research Laboratory (TRRL-Great Britain) and is described below.

Saturated specimens, approximately 100mm to 150mm in diameter and 1500mm high, are placed in an insulated cabinet. The tops of the samples are exposed to cold air at a temperature of approximately -17 degrees C and the bottoms of the samples are placed in contact with water, at a temperature of approximately 3 degrees C. As the samples freeze from top to bottom, frost heave is measured for 250 hours and is expressed as a rate or amount of heave per day. It should be recognized that the laboratory heave rate is an index test only rather than the rate to be experienced under field conditions.

---

(1) K.A. Linell et al., "Corps of Engineers' Pavement Design...," Highway Research Record, No. 33, 1963, p. 79, 94.

The Corps of Engineers uses the measured rate as an index of frost susceptibility as follows:

Average rate of heave/day (mm/day)	Frost susceptibility class
<0.5	Negligible
0.5 - 1.0	Very low
1.0 - 2.0	Low
2.0 - 4.0	Medium
4.0 - 8.0	High
>8.0	Very high

Swell Potential

In 1974, Gromko (1) suggested that the probable volume change due to the swelling of expansive clays may be estimated from gradation, the Atterberg Limits, and Standard Penetration Resistance, as shown in Table 2-4. The important gradation quantities are the percentage of soil passing through a No. 200 sieve (0.075mm) and the percentage which is smaller than 0.01mm i.e., colloid content. The important Atterberg Limits are the liquid, plastic, and shrinkage limits as well as the plasticity index. If it's important to measure swell characteristics directly, the free swell test that measures the percentage increase in volume of an air-dry clay after it has been immersed in water is recommended.

In 1969, Kassif et al. (2) described a series of tests that use the consolidation apparatus or similar equipment. The swelling percentage is the percentage increase in height of an air-dry consolidation sample when saturated with water. The swelling pressure is the pressure that must be exerted to keep the volume change at zero. By loading samples with various pressures and measuring the volume increase for each of them upon saturation, a series of intermediate points is determined, which suggest a curve similar to that shown in Figure 2-12.

In addition, the measured swell pressure will not attain a constant value until, perhaps, a few days after the sample is saturated. To shorten the test duration and to account for proving ring deflection, Lambe (3) has suggested the use of a standardized apparatus and procedure, in which the sample is allowed to swell for two hours and the proving ring is allowed to deflect. The Swelling Index is then defined as the pressure measured after two hours of swelling.

---

(1) G.J. Gromko, "Review of Expansive Soils," Journal of the Geotechnical Engineering Division, ASCE, Vol. 100, No. GT6, 1974, pp. 667-687.
(2) G. Kassiff, M. Livbeh, and G. Wiseman, Pavements on Expansive Clays, Jerusalem: Jerusalem Academic Press, 1969, 218 pp.
(3) T.W. Lambe, The Character and Identification of Expansive Soils, completed for the technical studies program of the Federal Housing Administration, 1960.

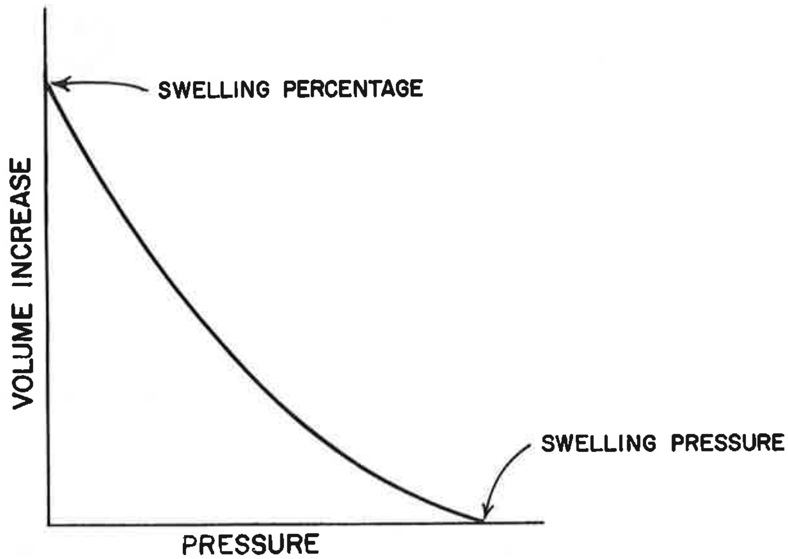


FIGURE 2-12. TYPICAL CURVE OF VOLUME INCREASE VERSUS PRESSURE OF SWELLING CLAY

Field Tests

The field tests used to determine soil stratigraphy and engineering soil parameters are described and evaluated in the following pages. Tests are grouped into direct field tests and geophysical or indirect tests.

Direct Methods

Interest in direct in-situ testing methods has greatly increased in recent years. The ASCE Specialty Conference on In-Situ Measurements of Soil Properties, 1975 provides numerous papers on the various methods. Other references on field testing methods include deMello (1) (for the standard penetration test), Schmertmann (2) (for the static cone penetration test), Bjerrum (3) (for the vane shear test), and Schmidt et al. (4) (for a review of all direct and indirect methods).

---

(1) V.F.B. deMello, "The Standard Penetration Test," Proceedings of the Fourth Pan American Conference on Soil Mechanics and Foundation Engineering, Vol. 1, 1971, pp. 1-86.
(2) J. H. Schmertmann, Guidelines for Cone Penetration Test Performance and Design, U.S. Department of Transportation, Federal Highway Administration, 1977, Report No. FHWA-TS-78-209, 145 pp.
(3) L. Bjerrum, "Embankments on Soft Ground," Proceedings of the Specialty Conference in Performance of Earth and Earth Supported Structures, Purdue University, Vol. 2, pp. 1-54.
(4) B. Schmidt, et al., Subsurface Exploration Methods for Soft Ground Rapid Transit Tunnels, U.S. Department of Transportation, Urban Mass Transportation Administration, Washington, D.C., 1976, Report No. UMTA-MA-06-0025-76-1.

The direct tests are able to test the soil mass in-situ, thus eliminating some sample disturbance effects. However, most engineers regard information obtained from classification, index, and engineering property tests on soil samples as most important, and would not rely strictly upon field tests for engineering analysis and design.

There are general difficulties associated with conducting all field tests, including hardware limitations, electronic or mechanical measurement errors, soil disturbance when the testing device is inserted in the ground, nonstandardized test procedures, and complicated boundary conditions that don't always conform to theories utilized when interpreting test results. As a result, many field tests are controversial, and some engineers regard them as nothing more than crude index measures of soil engineering parameters. Many field tests are still in the developmental stage and additional, carefully evaluated applications are necessary before they can be properly judged.

Table 2-5 lists direct measurement methods and the soil parameters measured by each. Table 2-6 summarizes the limitations, availability and use of each technique. Only those field tests that have been widely accepted in the engineering profession and that are judged practical for railroad problems have been considered.

Standard Penetration Test

The standard penetration test (SPT), one of the most frequently used direct methods of subsurface exploration in the United States, consists of driving a standard 2-inch O.D. split-barrel spoon sampler by means of a 140 pound weight falling 30 inches. The standard penetration resistance (N) is the number of blows required to drive the sample the last 12 inches of an 18-inch drive. Quite frequently the test is performed at 5-foot intervals or at stratum changes, although almost continuous profiles of SPT results may be obtained. (See Figure 2-13 for a diagram of the test.)

The SPT can be interpreted by readily available empirical correlations between the standard penetration resistance (N) and soil parameters such as consistency, relative density and shear strength. DeMello (1971) provides detailed discussion of these correlations which should be regarded only as approximations for a number of reasons. First, most empirical correlations are statistically derived from a broad but limited data base. Second, many correlations don't consider the effect of confining pressure, which may be especially important for granular soils.

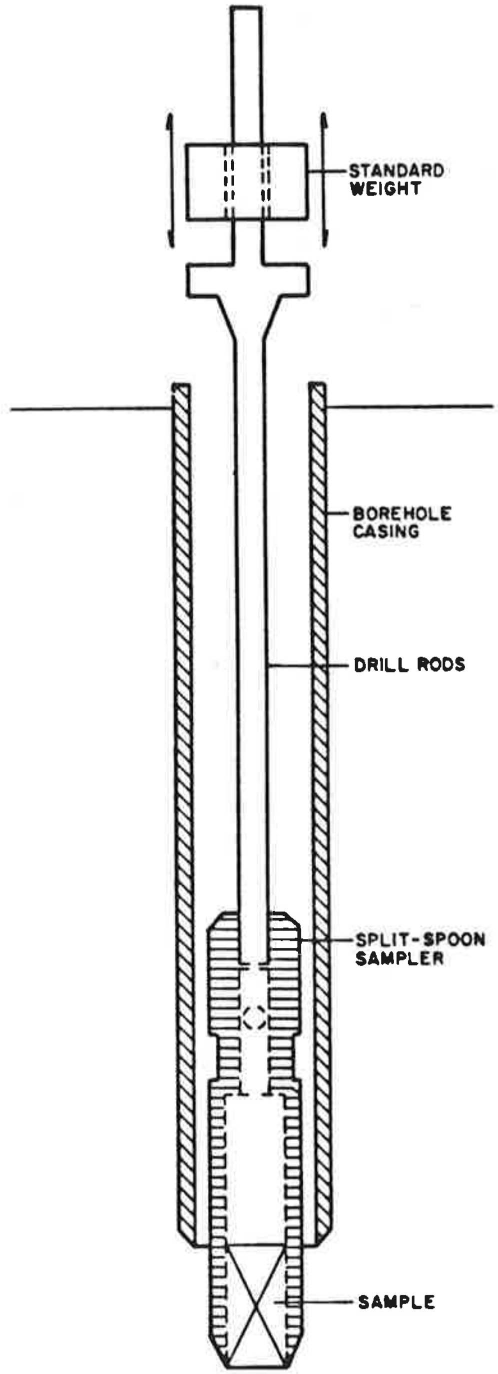
The procedure employed in performing the SPT is important. Proper seating of the sampler, connections between the drill rods, correct weight and drop, and proper washing out of the borehole before performing the test all have significant influence on the resulting N values. Partial obstructions such as cobbles may yield inordinately high blow counts yet remain undetected by an unqualified driller. Supervision by a qualified and experienced inspector is necessary for situations where accurate blow counts are critical.

TABLE 2-5. PARAMETERS MEASURED BY VARIOUS DIRECT IN-SITU TESTING METHODS

Type of Test \ Parameter	Shear Strength	In Situ State of Stress	Piezometric Head	Modulus of Deformation	Permeability	Soil Type
Standard Penetration Test	x					x
Static Cone Penetration Test	x					x
Dynamic Cone Penetration Test	x					x
Vane Shear Test	x					
Plate Bearing Test	x			x		
Piezometers		x	x		x	
Borehole Permeability Test					x	
Pressure-meter	x	x		x		

TABLE 2-6. LIMITATIONS, AVAILABILITY, AND USE OF VARIOUS DIRECT IN-SITU TESTING METHODS

Characteristics & Requirements / Type of Test	Hole Required: C-Cased O-Open S-Stabilized	Suitable for Analysis: C-Cohesive S-Fine Granular G-Coarse Granular	E-Empirical T-Theoretical	Reliability and Reproducibility	Approx. No. of Different Types	Approx. No. of Manufacturers	Comments
Standard Penetration Test	C or O or S	C, S, G	E	Fair	—	—	Common usage with test boring. Much experience. Several useful correlations.
Static Cone Penetration Test (Dutch Cone)	No hole	C, S	E & T	Good to Excellent	20	20	Recent developments with friction sleeves have led to ability to identify soil types. Very promising techniques.
Dynamic Cone Penetration Test	No hole	C, S, G	E & T	Fair to Good	8	8	Little American experience. Calculations yield values of N(SPT) or undrained shear strength.
Vane Shear Test	No hole	C	E & T	Good	3	5	Recent empirical corrections have improved analytical result.
Plate Bearing Test	—	C, S, G	E & T	Fair	—	—	Best suited to shallow soil conditions.
Piezometers	C or O or No hole	C, S, G	T	Excellent	16	26	Many variations available.
Borehole Permeability Tests	C or O	C, S, G	T	Poor to Good	—	—	Results general questionable. Need exists for standardization.



Reproduced from Subsurface Exploration Methods for Soft Ground Tunnels, p. 5-6, by B. Schmidt *et al.* Year of first publication: 1976.

FIGURE 2-13. STANDARD PENETRATION TEST

Cone Penetration Tests

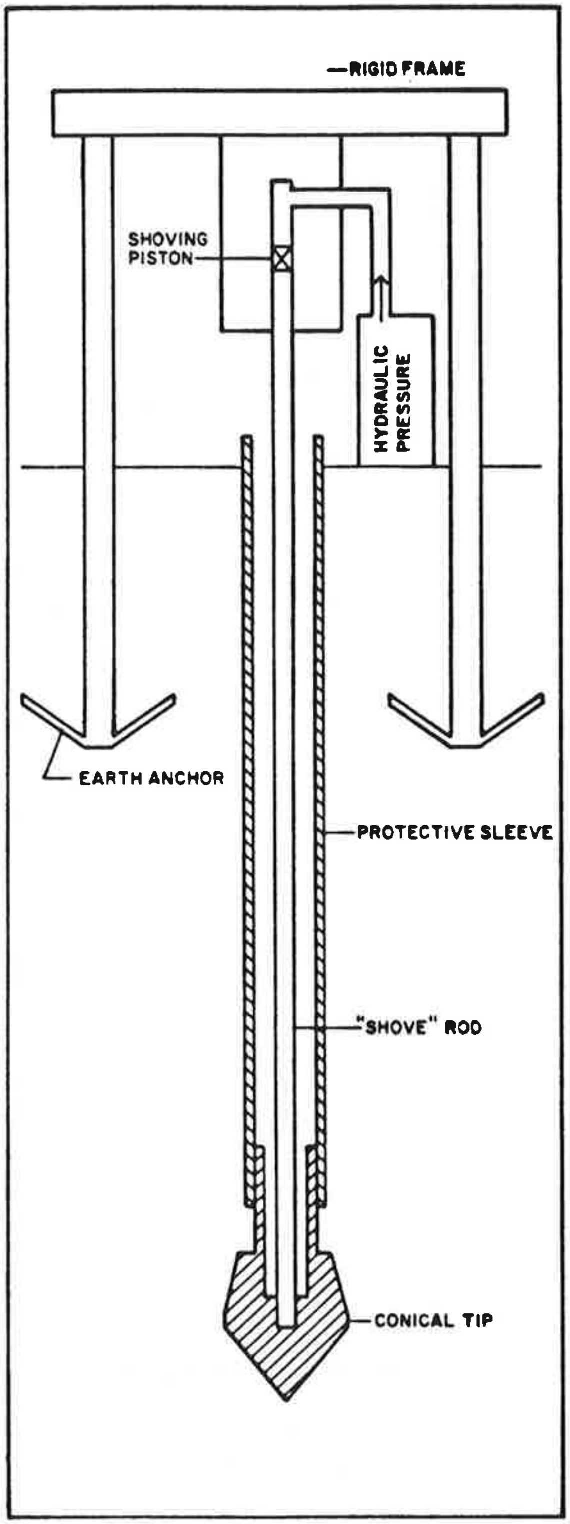
Static cone penetration tests (i.e., Dutch Cone tests) are performed by advancing a conical point at a constant rate through the substrata by using an hydraulic jack. The total resisting force necessary to maintain this constant rate of penetration is then measured. Cone penetrometers may be used in either sands or clays, above or below groundwater level, and can be advanced without a borehole in many cases. Cone penetration tests have been successfully performed through ballasted track by the U.S. Army Waterways Experiment Station (unpublished communication). At present, there is a variety of cone configurations in use throughout Europe, although a recently proposed ASTM specification (ASTM D-3441) calls for an apex angle of 60 degrees and a projected area of 10 square centimeters. European engineers have amassed and evaluated a large amount of static cone penetrometer data, much of which is reported in the authoritative text by Sanglerat (1). Figure 2-14 shows a diagram of the apparatus. Cone resistance is used to determine soil type and stratigraphy. Because data can be continuously recorded, thin layers of material can be detected that may significantly influence soil mass behavior. Recent improvements, summarized by Schmertmann in 1977, have focused on the development of a friction sleeve cone penetrometer which measures the end bearing force and the side frictional force (2). This permits determination of soil cohesion and/or friction angles, and hence the shear strength of sands and clays, by means of semi-empirical bearing capacity factors.

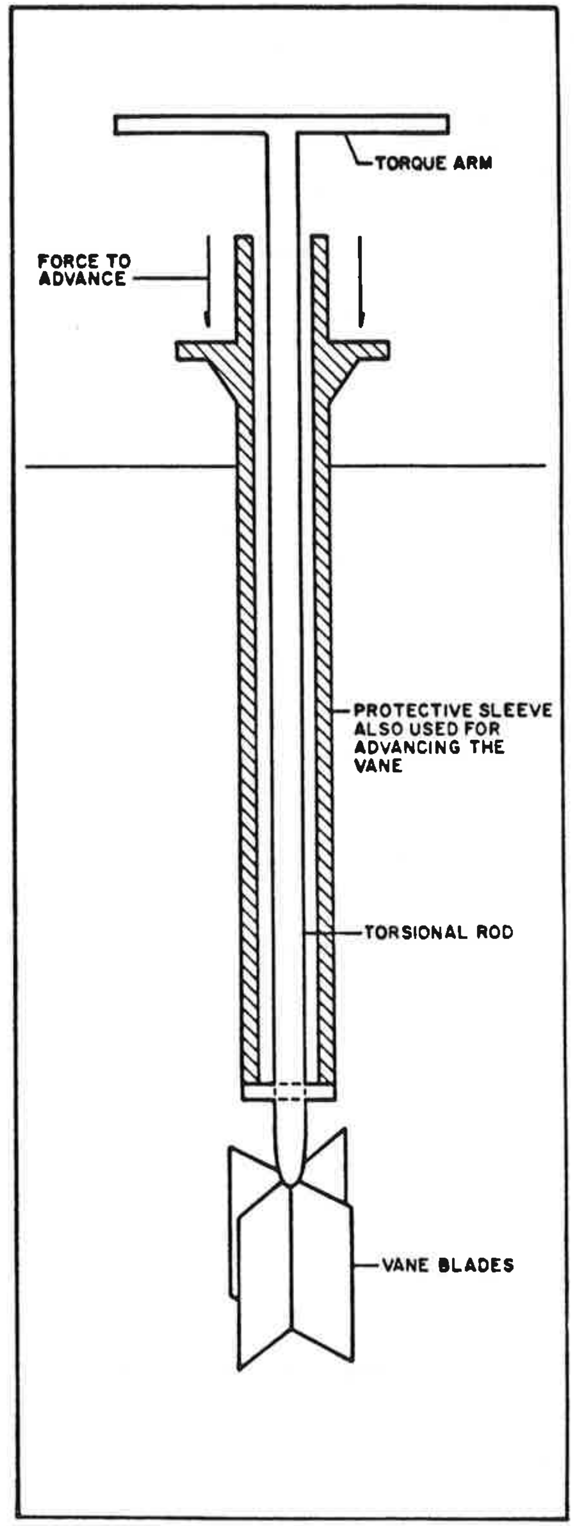
One major difficulty common to many field and laboratory tests is the dependency of the measured cone resistance on rate of penetration (or strain). The penetration rate interacts with the rate of excess pore pressure dissipation around the probe and with the time-dependent behavior of many soils (e.g., viscosity) to affect the measured cone resistance. To partially solve this problem, an advancement rate of 20 mm/sec is most often used. However, the basic problem of applying strength data obtained at one strain rate to the actual field problem where strains most likely occur more slowly still exists.

The cone penetration test may also be performed dynamically by driving a conical point by means of a falling weight, and noting the amount of advancement of the cone for each blow. The test yields basically the same type of information as the SPT, but it's not as well standardized and is less amenable to empirical correlation in the United States because of its lack of general use.

---

(1) G. Sanglerat, The Penetrometer and Soil Exploration, Elsevier Publishing Co., New York, 1972.
(2) J. H. Schmertmann, Guideline for Cone Penetration Test Performance and Design, U.S. DOT, Federal Highway Administration, 1977, Report No. FHWA-TS-78-209, 145 pp.





Reproduced from Subsurface Exploration Methods for Soft Ground Tunnels, p. 5-9, by B. Schmidt *et al.* Year of first publication: 1976.

[IGURE 2-14. STATIC CONE PENETRATION TEST   FIGURE 2-15. VANE SHEAR TEST

Field Vane Shear Test

The field vane shear test shown in Figure 2-15 consists of inserting steel vane blades into clay and applying a torque to rotate the vane at a constant rate until a shear failure occurs. Angular deformation and resisting torque are measured throughout the test. The vane apparatus may be advanced to large depths in soft soils without a borehole. In this case, however, protective sheaths are commonly placed around the probe rods in order to minimize torsional friction resistance on the rods. The vane is most commonly used in soft clays.

Hardware for the test is fairly well developed, with a variety of vane configurations. There is, however, a general need for more standardization. ASTM Standard D-2573 calls for a four-blade configuration with a height to diameter ratio of 2:1, consistent with the dimensions of many commercially available vanes.

Limitations of the vane include inordinately high-measured vane strengths if thin silt or fine sand seams are intersected in a clay soil. The stress system applied by the vane is unlike any prototype stress system, and theoretical interpretations should not be applied with great confidence. Finally, as for the static cone penetration test discussed previously, the vane is susceptible to strain rate effects. Therefore, vane strengths depend on the rate of rotation.

Plate Bearing Test

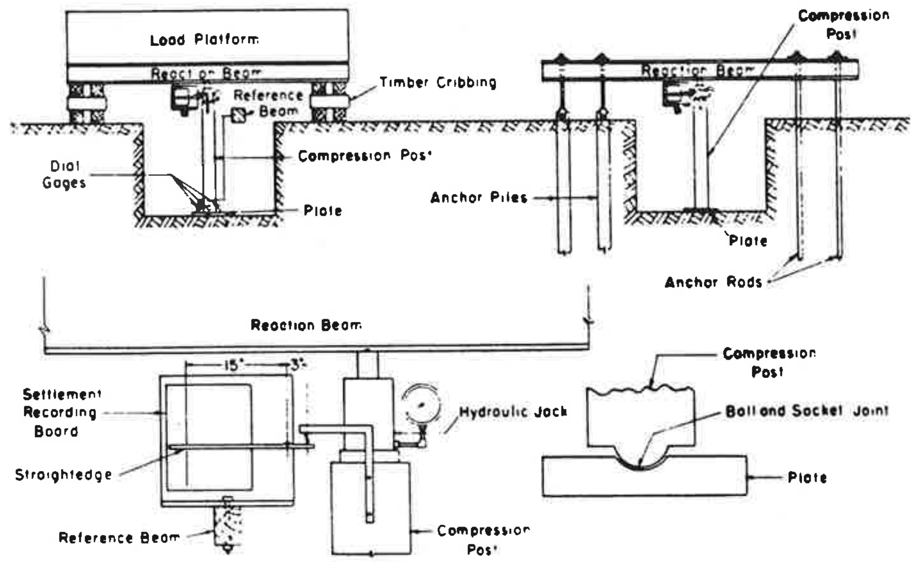
The Plate Bearing Test shown in Figure 2-16 involves loading the subgrade surface with a flat, steel plate. Usually, a 760-mm (30 inch) diameter plate is used to bear on the subgrade. Smaller plates are stacked between the bearing plate and the loading jack to increase the plate stiffness. In 1977, Selig et al. described the use of small and rapid plate bearing tests between or beneath railroad ties. (1)

The plate bearing test is most frequently used to measure settlements or stiffness, although it can also measure shear strength. The load versus settlement curve of a plate bearing test is commonly intepreted by means of closed form elastic solutions, empirical relationships between small bearing plates and large-scale footings, and finite element analyses for elastic and inelastic material.

The results of the plate bearing test are frequently expressed as a coefficient of subgrade reaction (CSR), which is the ratio of applied stress to resulting deflection in units of force per distance cubed, e.g., tons per cubic inch or newtons per cubic meter. Although the 760-mm plate is most commonly used, the subgrade modulus is usually quoted for a footing width or plate diameter of 305 mm (1 foot).

---

(1) E.T. Selig et al., Mechanics of Ballast Compaction: Field Measurements of Ballast Physical State Measurements, U.S. DOT, Transportation Systems Center, March 1982, pp. 71-95, Report No. FRA-ORD-81-16.2.



Reprinted, with permission, from the Annual Book of ASTM Standards, Part #19 (D1194). Copyright, American Society for Testing and Materials, 1916 Race Street, Philadelphia, Pennsylvania 19103.

FIGURE 2-16. TYPICAL SETUP FOR CONDUCTING STATIC LOAD TESTS

The coefficient of subgrade reaction is very convenient to use as input to various elastic analyses. However, CSR is a very poor representation of soil behavior, since it combines the independent effects of material properties (e.g., equivalent elastic modulus), and geometry (e.g., size, shape, and depth of the loaded area). From a soil behavior point of view, it's preferable to present the results of plate bearing tests in terms of equivalent moduli (Young's, shear, or constrained modulus) because these are material characteristics only.

Disadvantages of the plate bearing test include errors introduced by seating irregularities and lack of confinement and the fact that they are generally limited, by economics, to near surface soil deposits. Empirical or theoretical correction factors must be employed to correct for size and depth of loaded areas. In addition, the size and depth of the plate influences the thickness of the soil layer below the bearing level that significantly affects the plate settlement behavior. This is particularly important if the nature of the soil changes with depth. Errors will develop if the thickness of soil that is stressed in the plate bearing test isn't representative of subsurface conditions beneath the prototype structure. Despite these limitations, the plate bearing test--if carefully performed and interpreted--appears promising as a measure of shallow subgrade soils, ballast, and subballast behavior.

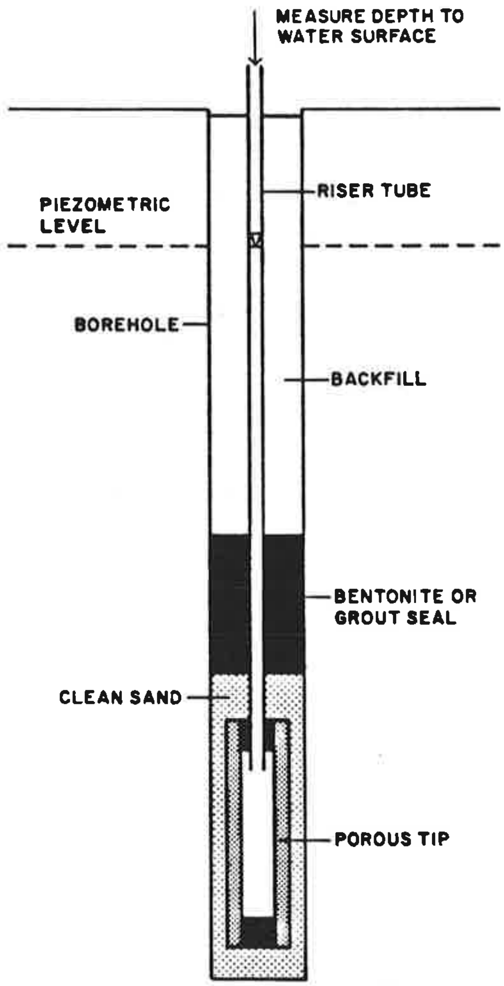
Piezometers

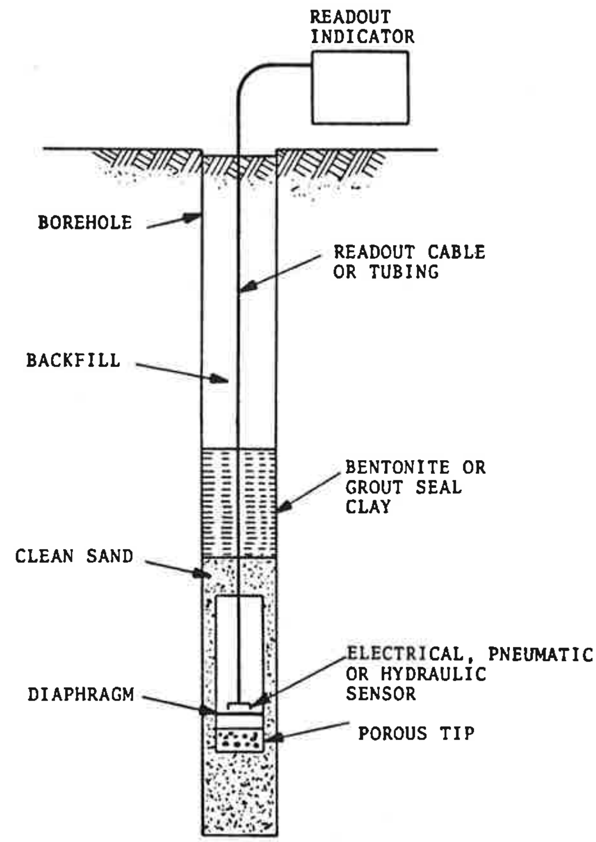
The use of piezometers, as shown in Figure 2-17 and 2-18, to determine static groundwater levels or excess pore water pressures is described in the following paragraphs. There is an abundant variety of available hardware, and proven methods are well known. In general, piezometers are categorized as flow piezometers or diaphragm piezometers.

Flow piezometers consist of a porous tip connected to a riser tube or standpipe. The porous tip is embedded in the ground at a desired location with the riser tube protruding to the ground surface. Water is free to flow through the porous tip and up the riser tube. The level obtained by the water in the riser tube represents the piezometric level at the tip. If the piezometric level is above the ground surface, the excess piezometric level is measured by means of a Bourdon gauge mounted directly on the riser at a convenient location.

If the riser tubes are exposed to freezing temperatures, extra precautions are required. Frequently, the riser tubes are filled with anti-freeze solutions. However, the measured water level must be adjusted for the altered density of the measuring fluid column. Diaphragm piezometers are often used when freezing is considered.

Diaphragm piezometers differ from flow piezometers in that a diaphragm separates the porewater from the pressure sensing arrangements. When under pressure, the diaphragm deflects slightly. The pressure may be determined by means of a resistance or vibrating wire strain gauge mounted on the diaphragm or by applying pneumatic or hydraulic pressure to the opposite side of the diaphragm. With any of these sensing systems, it's possible to obtain rapid readings of the existing piezometric head because the quantity of fluid flow required to activate the device is very low.





Reproduced from Subsurface Exploration Methods for Soft Ground Tunnels, pp. 5-18/19, by B. Schmidt et al. Year of first publication: 1976.

FIGURE 2-17. OPEN STANDPIPE PIEZOMETER

FIGURE 2-18. DIAPHRAGM PIEZOMETER

A recent innovation described in 1975 by Wissa et al. as "The Piezometer Probe" enables a diaphragm piezometer to be used as an indicator of both pore pressure and permeability, and possibly of consistency and relative density. The probe consists of an electrical pore pressure transducer pushed into the ground at a constant rate. Hydrodynamic response is rapid, and pore pressures generated during pushing can then be measured. Changes in the measured pressure can be used to identify qualitatively the consistency or relative density of the soil being penetrated. A diagram of the probe is shown in Figure 2-19.

Groundwater observations are recommended because in almost all situations (cuts, fills, granular soils, fine grained soils) the location of the groundwater table will impact considerably on the mechanical and environmental performance characteristics of the subgrade soils. Observations may be made during boring operations or at the completion of borings. Water levels observed in this manner may reflect the influence of the boring procedure and not other important factors such as seasonal or tidal fluctuations in water level. Preferably, piezometers or slotted pipe should be installed in completed boreholes to provide more accurate long term observations of groundwater level.

Borehole Permeability Test

Borehole permeability tests are performed by pumping water either out of or into a borehole. The test may be performed as a constant head test, in which the rate of pumping necessary to maintain a constant piezometric level in the borehole is measured. Alternatively, variable head conditions may be obtained by observing the change in piezometric level in the borehole after pumping is stopped.

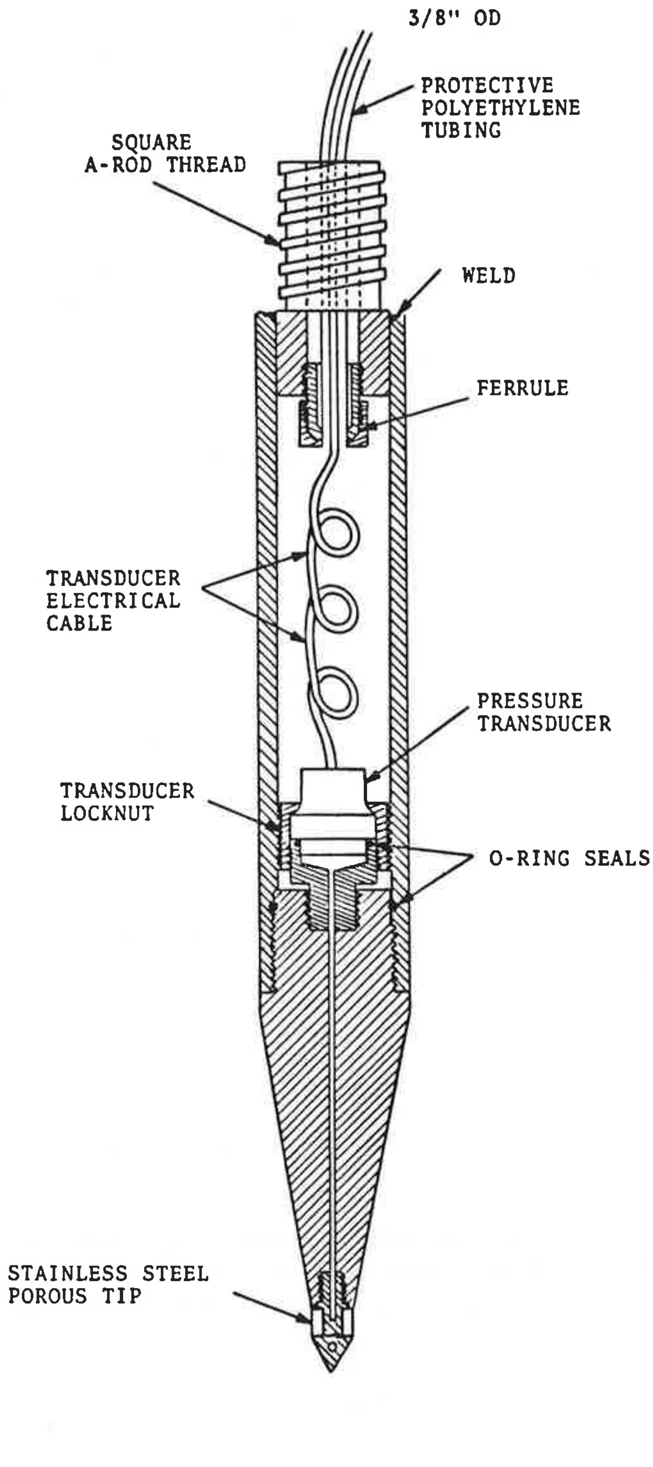
The permeability of a localized zone of soil surrounding the borehole is computed by applying well-established, theoretically derived equations, that were summarized by Hvorslev (1) and which account for the type of test and imposed boundary conditions (e.g., flow through bottom face of casing only or flow through an extended pervious zone below casing).

The exfiltration test (i.e., out of borehole) is used more often than the drawdown test (i.e., into a borehole) because surface pumps limit the drawdown to a maximum of 5m to 7m. Constant head tests are preferred to variable head tests because they are easier to perform properly, and generally provide more reliable and consistent data.

One major limitation of borehole permeability tests is the possibility that the limited zone of tested soil may not represent overall soil conditions. The effects of very thin, but important, nonrepresentative layers of soil (e.g., silt layers within a primarily clay soil) may be masked. The most serious other problem in connection with exfiltration tests is the development of a silt cake on the face of the diffusion zone that impedes flow out of the borehole. Because of this it is preferable to have an extended pervious zone below the casing, rather than relying solely upon flow through the horizontal plane at the casing bottom.

---

(1) M.J. Hvorslev, Time Lag and Soil Permeability in Groundwater Observations, U.S. Army Waterways Experiment Station, Bulletin No. 36, 1951.



Reproduced from Subsurface Exploration Methods for Soft Ground Tunnels, p. 5-19, by B. Schmidt *et al.* Year of first publication: 1976.

FIGURE 2-19. PIEZOMETER PROBE

Because of these limitations, the results of borehole permeability tests are often inconsistent or are not representative of the true in-situ permeability. The severity of these problems depends, to a large degree, on the skill and experience of the driller making the boring and performing the test. Where any questions remain, the most accurate means of determining permeability is by means of a field pumping test with an accompanying measurement of drawdown in nearby groundwater observation wells.

Simple exfiltration tests in auger holes or test pits may be used to approximate the permeability of shallow granular deposits. These tests, referred to as percolation tests, are commonly performed when designing sanitary leaching fields. They are commonly performed as falling head tests, although the U.S. Bureau of Reclamation Earth Manual, 1974 describes the equipment and procedures for more elaborate constant head tests. ASTM Method D-3385 has standardized a constant head infiltration test shown in Figure 2-20.

Percolation tests are subject to the limitations of the borehole permeability tests described previously. Despite these limitations, percolation tests provide crude index measures of shallow subgrade soils and permeability. If performed in the ballast or subballast, they may also measure the degree of ballast permeability loss due to fouling.

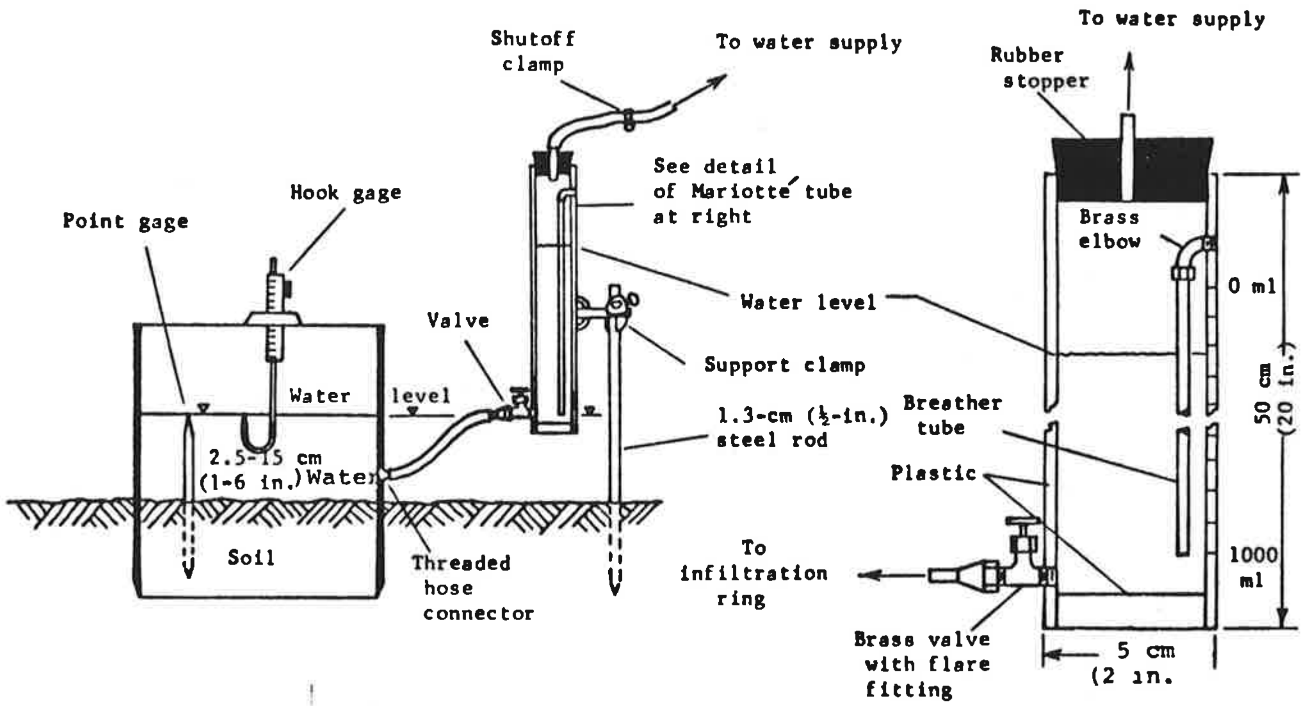
Geophysical Methods

Surface seismic and electrical resistivity surveys are thought to be the geophysical techniques that are best suited for railroad engineering. These two methods are described and evaluated in the following paragraphs.

Seismic Refraction and Reflection Surveys--These are conducted by introducing energy into the ground and observing arrival time and ground motion at one of a series of detectors placed at increasing distances from the point where energy is introduced. Explosives are commonly used as a source of the energy, and detection of the resulting seismic waves is accomplished by recording the amplified electric outputs of small coil-magnet geophones which respond to relative motion between the soil and the suspended magnet. Many types of mechanical energy sources are also used.

As an exploration technique in engineering application, the seismic method detects interfaces of strata and also seismic wave velocity. The latter generally increasing with density and stiffness is an index of material type. For example, in refraction surveys, the wave velocity of soft clay is lower than that of dense sand and gravel; that of igneous rock still higher.

Energy introduced near the surface travels away from the starting point in the form of seismic waves, and these are refracted and reflected at subsurface boundaries between different materials. The seismic waves considered in refraction surveys are those that are refracted along subsurface boundaries. Likewise, seismic waves that are reflected from subsurface boundaries are of interest in reflection surveys.



Reproduced, with permission, from the Annual Book of ASTM Standards, Part #19 (D3385). Copyright, American Society for Testing and Materials, 1916 Race Street, Philadelphia, Pennsylvania 19103.

FIGURE 2-20. ASTM INFILTRATION (PERCOLATION) TEST

Surface seismic testing techniques have the following limitations:

a. In seismic refraction, the presence of a low-velocity layer beneath higher-velocity materials causes seismic waves to refract energy downward, resulting in the layer not being represented. A common example of a low-velocity layer would be loose river gravels beneath dense clay strata, or a layer of organic silt beneath sands and gravels.

b. Survey of shallow reflection horizons in the subsurface is complicated by direct and refracted waves reaching the geophones before the reflections. The early high amplitudes commonly mask shallow reflection signal arrivals, severely limiting the "shallowness" of effective surveying.

c. All of the seismic techniques depend upon a person's ability to notice and accurately time seismic wave arrivals at the detectors. Effects on signal detectability by natural and man-made earth noise, geometrical energy spreading, wave reflection coefficients, mode conversions, and energy dissipation must be considered.

d. Resolution of subsurface discontinuities by seismic methods requires sufficient energy to overcome any factors that reduce signal amplitudes. A large amount of energy is required to achieve deep penetration, yet energy sources must be environmentally acceptable.

Surface Electrical Resistivity Surveys - Soils tend to have characteristic electrical properties in localized areas, which can often be mapped by geoelectric methods. The presence and electrical characteristics of subsurface water are also detected by using direct electrical techniques. Profiles of the thicknesses of subsurface units with contrasting electrical properties, depths to ground water tables, and lateral changes in elevation of the different subsurface units are typical results of such surveys.

Resistivity surveys are conducted by inserting electrodes into the ground at a depth of about 300mm, applying electrical power to the electrodes, and measuring the potential in the ground at positions away from the powered electrode positions. Analysis of the measurements is based upon the assumption that the voltage at the measurement points is influenced by deeper materials as distance from the powered electrodes increases. A decrease in electrical power with distance from the powered electrodes results from resistance to current flow in the subsurface materials. Changes in the trend with increasing distance are used to indicate the depth where the change occurs, and the amount of changes in the trend indicates the change in resistance characteristics.

Discussion - The following is a listing of the advantages and disadvantages to using geophysical methods for railroad projects:

a. Highly trained personnel are ordinarily required to lay out and interpret a geophysical survey

b. Precise measurements are important for all methods, but accuracy in the interpretation and inferences drawn from the measurements depend very much on the experience of the interpreter

c. All methods present the "averaged" effects of materials between and around sources and points of observation

d. Geophysical explorations must be accompanied by direct explorations such as borings and test pits

e. The outstanding advantage of surface seismic refraction and resistivity surveys is that they provide pertinent subsurface information along a line, quickly and economically. In addition, they are the geophysical techniques most familiar to civil engineers.

2.4 SOIL CLASSIFICATION SYSTEMS

Soil classification systems provide a uniform and concise vocabulary for describing soils, and a categorization of soils into groups, each with similar performance characteristics. Most classification systems are based upon simple visual, manual, or laboratory index tests, which provide a rapid and usually inexpensive (albeit approximate) indication of engineering performance. Many of the commonly used classification systems summarize general engineering performance characteristics by means of "Engineering Use Charts," some of which are presented in Appendix A.

The methods of classifying soils are:

a. Grain size scales--Verbal description of soil particle size based upon an arbitrary grain size scale

b. Textural--Based upon grain size distribution

c. Visual-manual--Based upon visual examination and simple manual tests

d. Engineering Use--Based upon texture and plasticity with grouping formulated according to general engineering performance

e. Geologic--Based upon geologic history or origin of soil deposits

f. Agricultural--Based upon a study of shallow soil profiles with major divisions dependent on climatic and drainage conditions as they influence agricultural use.

Casagrande (1) and Yoder (2) provide comprehensive summaries of the many classification systems available, and these writings serve as a basis for this discussion. All classification systems are limited because they are based upon visual and manual inspection and/or simple tests on disturbed samples rather than the undisturbed soil mass in-situ, and because the range of engineering performance characteristics for any grouping of soils may be quite large.

---

(1) A. Casagrande, "Classification and Identification of Soils," Transactions, ASCE, 1948, pp. 901-992.
(2) E.J. Yoder, Review of Soil Classification Systems Applicable to Airport Pavement Design, prepared for the U.S.D.O.T. Federal Aviation Administration, FAA-RD-73-169, 1974, 123 pp., (AD-783 190).

In the following sections of this chapter, grain size, textural, visual-manual, and engineering use classification systems that are most applicable to railroad engineering will be discussed and evaluated.

Grain Size Scales

Grain size classification involves the division of grain sizes into groupings of gravel, sand, silt, and clay on the basis of an arbitrary numerical scale. Some of the many grain size scales that have been used are compared in Figure 2-21.

The major disadvantage of grain size scale classification is that engineering performance of silt and clay size particles is dependent upon many other material characteristics. Also, the lack of standardization between the various scales creates ambiguity, so that the scale being used should be clearly stated. The M.I.T. classification system appears to be used most often by engineers worldwide, and is the recommended grain size scale for railroad engineering.

Textural Soil Classification

Textural classification systems categorize soils regarding the percentages of gravel, sand, silt, and clay particles within the soil sample. Textural systems are simple to use and may indicate, fairly precisely, the performance characteristics of coarse-grained soils. Of the many textural systems available, the triangular chart and Burmister systems are best suited for railroad applications, and will be described in the following paragraphs.

Triangular chart systems classify soils according to location on a standardized triangular chart as determined by the percentages of sand, silt, and clay size constituents. Classification applies only to the portion of soil that is smaller than gravel. Because of this, the system should not be used for soils having large amounts of gravel. Triangular charts used by the United States Bureau of Public Roads and the Lower Mississippi Valley Division of the Corps of Engineers are presented in Figure 2-22. The difficulty with these charts is that the classification does not reflect the plasticity of fine-grained soils.

The Burmister System (1) is illustrated in Tables 2-7 and 2-8. Table 2-7, for fine-grained soils, is based on soil plasticity rather than texture. Table 2-8, for coarse-grained soils, is based entirely upon grain size distribution. The Burmister System for identification of granular soils, is relatively easy to use and provides accurate, precise classification and descriptions of soils.

---

(1) D.M. Burmister, "Identification and Classification of Soils: An Appraisal and Statement of Principles," Proceedings, Symposium on Identification and Classification of Soils, ASTM STP 113, 1950, pp.3-24.

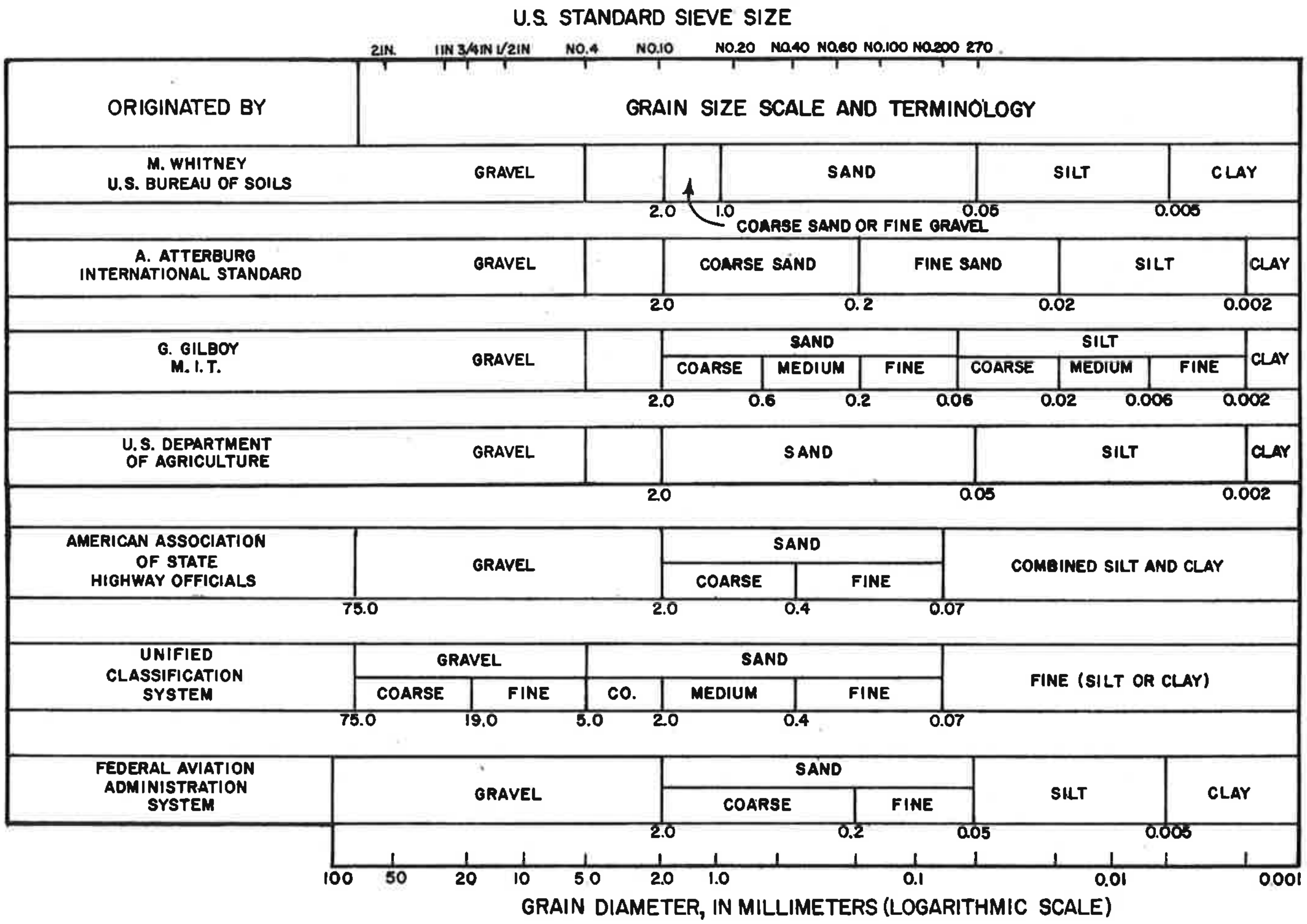
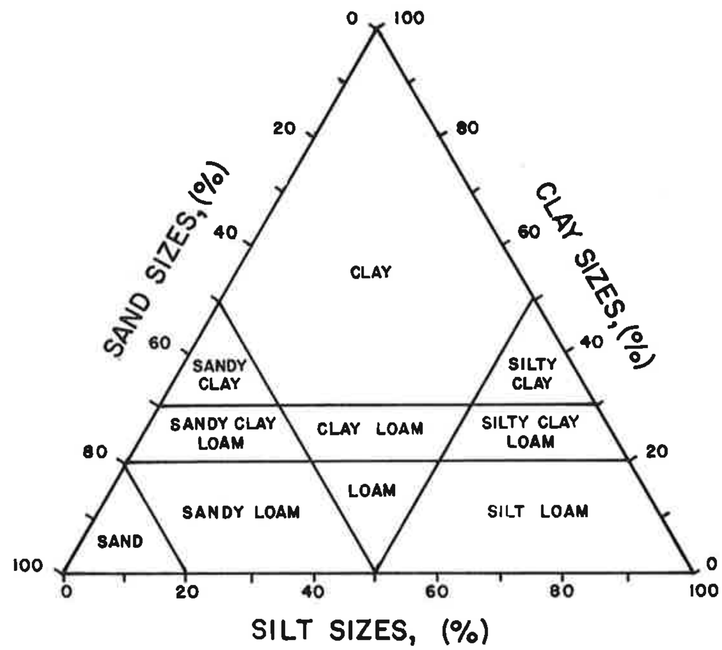


FIGURE 2-21. COMPARISON OF PRINCIPAL GRAIN SIZE SCALES

(a) U.S. BUREAU OF PUBLIC ROADS



(b) LOWER MISSISSIPPI VALLEY DIVISION, U.S. ARMY CORPS OF ENGINEERS

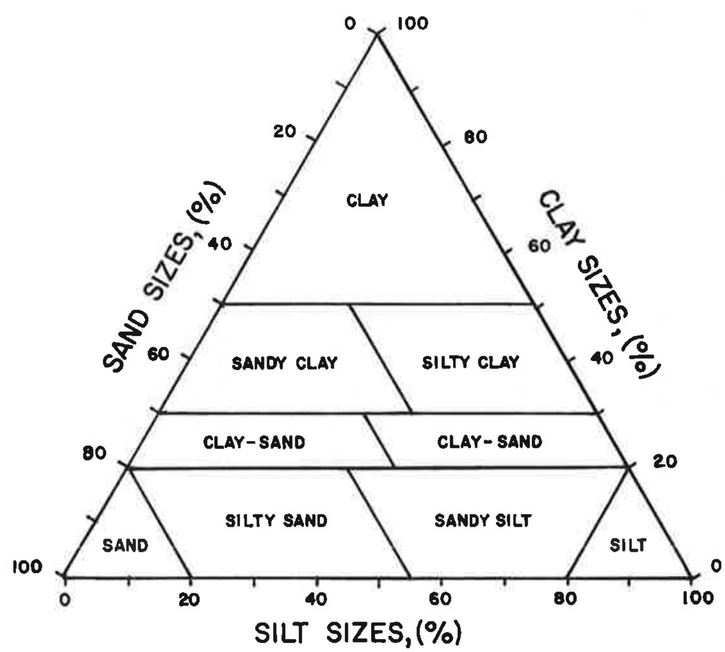


FIGURE 2-22. SOIL CLASSIFICATION TRIANGLES

TABLE 2-7. DEFINITIONS OF TERMS FOR THE IDENTIFICATION OF COMPOSITE CLAY SOILS ON AN OVERALL PLASTICITY BASIS AFTER BURMISTER, 1950

Degree of Overall Plasticity,[a] PI'		Overall Plasticity Index, PI' Sand-Silt-Clay Components	Identification Principal Component Written Symbol		Identification Minor Component Written Symbol		Smallest Diameter of Threads, in.
Nonplastic.........		0	SILT	$\bar{S}$	Silt	$\bar{S}$	None
Slight	S......	1 to 5	Clayey SILT	CY$\bar{S}$	Clayey Silt	CY'$\bar{S}$'	1/4
Low	L......	5 to 10	SILT & CLAY	$\bar{S}$-C	Silt & Clay	$\bar{S}$-C	1/8
Medium	M......	10 to 20	CLAY & SILT	C-$\bar{S}$			1/16
High	H......	20 to 40	Silty CLAY	$\bar{S}$YC			1/32
Very High	VH.....	40 and greater	CLAY	C			1/64

[a]Closer designations may be made, if considered significant, of plasticity by the use of the plus (+) and minus (-) signs immediately following the plasticity term.

Symbol designations of plasticity, where used, should be written as follows, for example:

Slight- Plasticity,	S-PI'
Low Plasticity,	LPI'
High+ Plasticity,	H+PI'

TABLE 2-8. DEFINITIONS OF TERMS FOR THE IDENTIFICATION OF GRANULAR SOILS AFTER BURMISTER, 1950

(a) Definition of Terms Identifying the Composition of Granular Soils						(b) Definition of Terms Identifying the Gradation of Granular Component		
Component	Identification		Proportion Terms[a]		Defining Ranges of Percentages by Weight	Gradation Designations for Identification		Defining Proportions
	Written	Symbol	Written	Symbol		As Written	Symbol	
Principal Component	GRAVEL	G				coarse to fine	cf	All fractions greater than 10 percent of the component, but the medium fraction predominates[b]
	SAND	S			50 or more	or coarse to medium to fine	cmf	
	SILT	$\bar{S}$				coarse to medium	cm	Less than 10 percent fine
Minor Component	Gravel	G	and	a.	35 to 50	medium to fine	mf	Less than 10 percent coarse
	Sand	S	some	s.	20 to 35	medium	m	Less than 10 percent coarse and fine
	Silt	$\bar{S}$	little	l.	10 to 20	fine	f	Less than 10 percent coarse and medium
			trace	t.	1 to 10			

(c) Definitions of Identifying Soil Characteristics

General color of the whole soil, preferable moist—gray (g), brown (b), yellow-brown (yb), red-brown (rb), light gray (lg), medium gray (mg), dark gray (dg), etc.
Maximum particle size of gravel.
Predominating grain shape—water-worn, subangular, angular, scale-like, etc.
Kind of rock or mineral—granite, limestone, shale (fragments), and quartz, feldspar, etc.
Hardness, soundness, or friable condition.
Constituents—mica, shells, roots and humus, organic matter, foreign matter (brick, glass, concrete, etc.), etc.

(See Footnotes to Table 2-8, p. 66)

FOOTNOTES TO TABLE 2.8

[a]Proportions refer to percentages of the whole soil finer than and coarser than the principal component. Closer designations of proportions may be used, if considered significant, particularly for the gravel component, as follows:

Plus	(+)	Nearer the upper limit of a proportion
Minus	(-)	Nearer the lower limit of a proportion
No Sign		Middle range of a proportion

For example: "some-" nearer 20 percent
"little+" nearer 20 percent

[b]The predominating fraction, especially for the gravel component, may be designated by adding a plus (+) sign immediately following the fraction term, if considered significant.

For example: "coarse+ to fine Gravel"
"medium to fine+ Sand"

Visual-Manual Descriptive Systems

Many simple visual and manual tests have been developed to aid in a rapid and consistent description of soils. These procedures are used in the field or laboratory during inspection of soil samples when more elaborate and time-consuming classification tests are not practical. Visual and manual tests only approximate performance characteristics and should be supplemented and confirmed by more elaborate index property tests.

Visual-manual procedures have been standardized by ASTM (D2488) and are summarized in Table 2-9 through 2-11. They have been incorporated into the Unified Classification System described further in this section.

Table 2-9 indicates that, for coarse-grained soils, the classification is based primarily on a visual observation of constituent grain size distribution, grain shape, color, and structure. Classification of coarse grained soils is accomplished best when the sample is spread out in a pan or on a cloth. For field identification, fines are those particles that are indistinguishable to the naked eye.

Tables 2-10 and 2-11 illustrate that, for fine-grained portions of a soil, classification is based upon the dry strength, dilatency, and plastic thread manual procedures described in ASTM D2188 as follows:

1. Select a representative sample of the material for examination and form cubes approximately 13mm (1/2 inch) in size after the gravel and coarse sand fraction has been removed.

2. Mold one of the cubic samples until it has the consistency of putty, adding a small amount of water if necessary. Allow the sample to dry completely, and then test the strength of the dry sample by crushing it between fingers. Resistance by dried soil to crushing grows with increasing plasticity.

3. Add sufficient water, if necessary, to the other sample to produce a soft, but not sticky, consistency. Smooth the soil pat in the palm of one hand with the blade of a knife or a small spatula, shake horizontally, and strike the back of the hand vigorously against the other hand several times. Squeeze the sample by closing hand. A silty soil will readily bleed water to the surface when shaken and will absorb the water when squeezed; clay soils bleed and absorb water very slowly.

4. Shape the sample into an elongated pat and roll it by hand on a smooth surface or between the palms into a thread about 3mm (1/8 inch) in diameter. Reroll repeatedly until the thread crumbles at a diameter of about 3mm. The thread will crumble near the plastic limit. Highly plastic clays become very stiff as they approach the plastic limit; silty soils become friable as they approach the plastic limit.

Visual-manual classification procedures are simple to use and repeatable, if performed by experienced personnel. Individuals should check their classifications periodically and compare with numerical laboratory test results.

TABLE 2-9. CHECK LIST FOR DESCRIPTION OF COARSE-GRAINED SOILS

1. *Typical Name* Boulders Cobbles Gravel Sand
   Add descriptive adjectives for minor constituents.
2. *Gradation* Well graded Poorly graded (Uniformly graded or Gap-graded)
   Describe range of particle sizes or predominant size or sizes as coarse, medium, or fine sand or gravel.
3. *Maximum Particle Size* Note percent boulders and cobbles
4. *Size Distribution* Approximate percent gravel, sand and fines in fraction finer than 3 in. (76 mm). Indicate plasticity of fines (See 7.5).
5. *Grain Shape* Angular Subangular Subrounded Rounded
6. *Mineralogy* Rock type for gravel, predominant minerals in sand.
   Note especially presence of mica flakes, shaly particles and organic material.
7. *Color* Use Munsell notation, if possible
8. *Odor* None Earthy Organic
   May be neglected except for dark colored soils.
9. *Moisture content* Dry Moist Wet Saturated
10. *Natural Density* Loose Dense
11. *Structure* Stratified Lensed Nonstratified
12. *Cementation* Weak Strong
    Note reaction with HCl as none, weak or strong.
13. *Local or Geologic Name*
14. *Group Symbol* Estimate if desired. See Classification Chart, Fig. 1, ASTM Method D 2487.

TABLE 2-10. IDENTIFICATION OF FINE-GRAINED SOIL FRACTIONS FROM MANUAL TEST

Typical Name	Dry Strength	Dilatancy Reaction	Toughness of Plastic Thread	Plasticity[a] Description
Sandy silt	none—very low	rapid	weak—soft	none—slight
Silt	very low—low	rapid	weak—soft	none—slight
Clayey silt	low—medium	rapid—slow	medium stiff	slight—medium
Sandy clay	low—high	slow—none	medium stiff	slight—medium
Silty clay	medium—high	slow—none	medium stiff	slight—medium
Clay	high—very high	none	very stiff	high
Organic silt	low—medium	slow	weak—soft	slight
Organic clay	medium—very high	none	medium stiff	medium—high

[a] The term low may be substituted for slight in the description of plasticity.

TABLE 2-11. CHECK LIST FOR DESCRIPTION OF FINE-GRAINED AND PARTLY-ORGANIC SOILS

1. *Typical Name* Sandy Silt Silt Clayey Silt Sandy Clay
   Silty Clay Clay Organic Silt Organic Clay
2. *Maximum Particle Size* Note percentage of boulders and cobbles
3. *Size Distribution* Approximate percent gravel, sand and fines in fraction finer than 3 in. (76 mm)
4. *Dry Strength* None Very Low Low Medium High Very High
5. *Dilatancy* None Slow Rapid
6. *Plastic Thread* Weak and Soft Medium Stiff Very Stiff
7. *Plasticity of Fines* None Slight (low) Medium High
8. *Color* Use Munsell notation, if possible. Note presence of mottling or banding.
9. *Odor* None Earthy Organic
   May be neglected except for dark-colored soils.
10. *Moisture Content* Dry Moist Wet Saturated
11. *Consistency* Soft Firm (Medium) Stiff Very Stiff Hard
12. *Structure* Stratified Laminated (Varved) Fissured
    Slickensided Blocky Lensed Homogeneous (Nonstratified)
13. *Cementation* Weak Strong
    Note reaction with dilute hydrochloric acid as none, weak or strong.
14. *Local or Geologic Name*
15. *Group Symbol* Estimate if desired. See Classification Chart, Fig. 1, ASTM Method D 2487

Engineering Classification Systems

Engineering classification systems are based upon texture and plasticity, with soils grouped according to engineering performance. These classifications are related to design procedures for a specific application (e.g., highways or airfields) or to Engineering Use Charts, which are more general.

The three most widely used engineering soil classification systems used in civil engineering are the Federal Aviation Agency system (FAA), the American Association of State Highway and Transportation Officials system (AASHTO), and the Unified Soil Classification system (USC), developed originally by Casagrande for the U.S. Army Corps of Engineers and Bureau of Reclamation.

Federal Aviation Agency System - The FAA classification system as outlined in Table 2-12 and Figure 2-23 is used in conjunction with a thickness design method for airfield pavements. The classification procedure is based on grain size characteristics of soil that is finer than the No. 10 sieve (2mm). Categories are determined by the grain size as separated on the No. 60 (0.25mm) and the No. 270 (0.05mm) sieves as well as the Atterberg limits of the fine-grained portion. The final soil group rating of the subgrade is used to determine pavement thickness; the system primarily reflects the influence of the fine-grained soil fraction on the performance of pavement subgrades.

American Association of State Highway and Transportation Officials System - In the AASHTO system, as outlined in Table 2-13, classification of coarse-grained and fine-grained material is based on the amount of material passing the No. 200 sieve (0.075mm). Soil with 35 percent or less, by weight, passing through this sieve is designated as coarse grained; soil with more than 35 percent passing through the sieve is designated as fine grained. Plasticity tests are performed on that portion of the material that passes through the No. 40 sieve (0.425mm). The AASHTO system provides additional precision in classifying soil by means of a group index which is a term based on the amount of soil passing the No. 200 sieve and plasticity limits of the soil.

Unified Soil Classification System - The Unified System is outlined in Table 2-14. Classification of soils using the USC system can be based on laboratory gradation and Atterberg limits tests or visual-manual examination. The system divides soils into two broad classes, coarse-grained and fine-grained soils, which are determined by whether more or less than 50 percent of the material passes the No. 200 sieve. Coarse-grained soils are divided into sand and gravel, the distinction made as finer or coarser than the No. 4 sieve (4.75mm). Fine-grained soils are divided into silt and clay, the distinction being based on plasticity limits. Organic soils are also divided by plasticity limits.

TABLE 2-12. FAA SOIL CLASSIFICATION

Soil Group		Mechanical Analysis				Liquid Limit	Plasticity Index
		Material Retained on No. 10 Sieve—Percent[1]	Material Finer Than No. 10 Sieve—Percent				
			Coarse Sand Passing No. 10 Retained on No. 40	Fine Sand Passing No. 40 Retained on No. 200	Combined Silt and Clay Passing No. 200		
Granular	E-1	0-45	40+	60-	15-	25-	6-
	E-2	0-45	15+	85-	25-	25-	6-
	E-3	0-45	-----	-----	25-	25-	6-
	E-4	0-45	-----	-----	35-	35-	10-
Fine Grained	E-5	0-55	-----	-----	45-	40-	15-
	E-6	0-55	-----	-----	45+	40-	10-
	E-7	0-55	-----	-----	45+	50-	10-30
	E-8	0-55	-----	-----	45+	60-	15-40
	E-9	0-55	-----	-----	45+	40+	30-
	E-10	0-55	-----	-----	45+	70-	20-50
	E-11	0-55	-----	-----	45+	80-	30+
	E-12	0-55	-----	-----	45+	80+	-----
	E-13	Muck and Peat—Field Examination					

[1]If percentage of material retained on the No. 10 sieve exceeds that shown, the classification may be raised, provided such material is sound and fairly well graded.

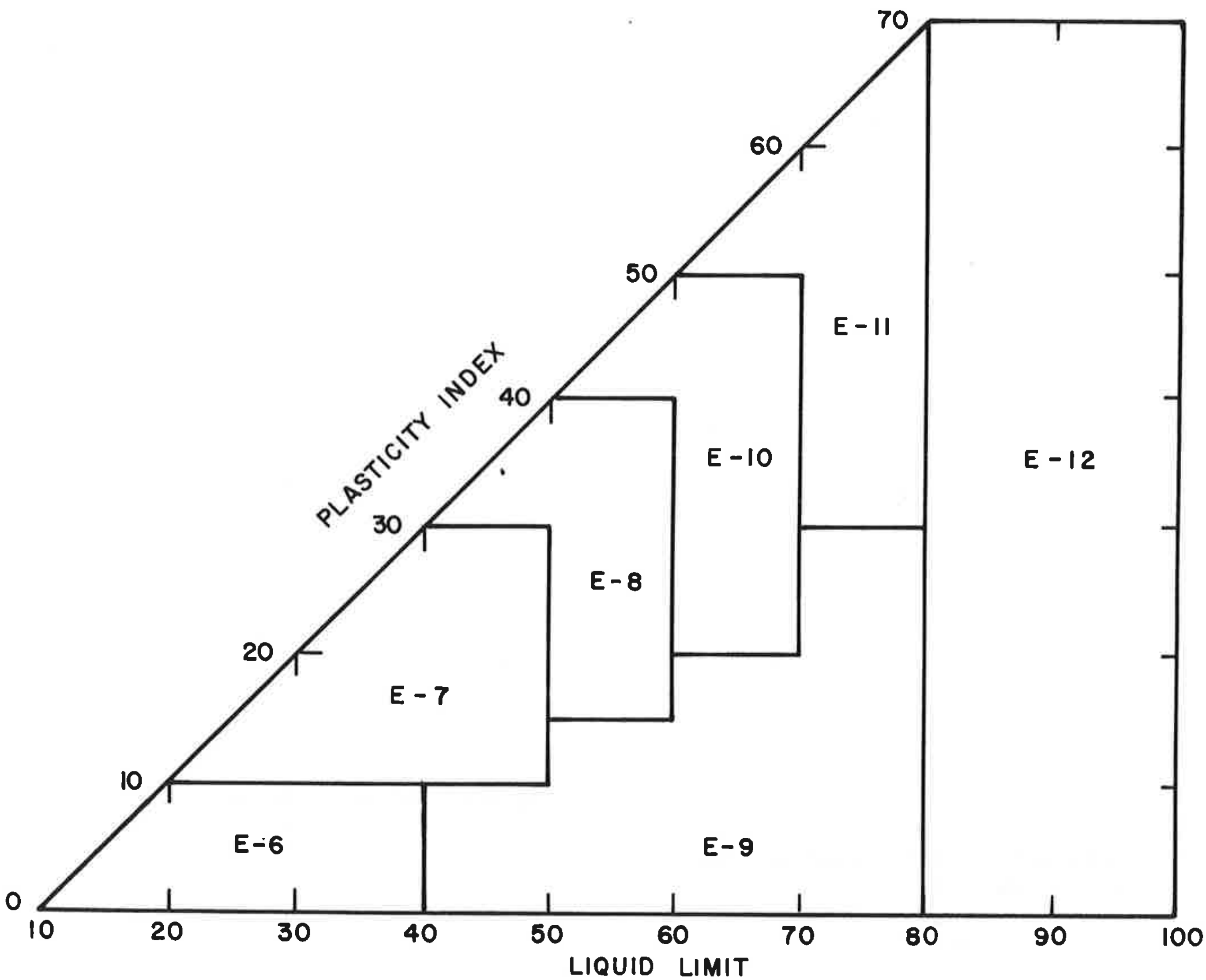


FIGURE 2-23. FAA SOIL CLASSIFICATION SYSTEM FOR FINE-GRAINED SOILS

TABLE 2-13. CLASSIFICATION OF SOILS AND SOIL-AGGREGATE MIXTURES BY AASHTO

	Granular Materials (35% or less passing 0.075 mm)		
Group Classification	A-1	A-3[a]	A-2
Sieve Analysis, Percent Passing:			
2.00 mm (No. 10)	....	....	....
0.425 mm (No. 40)	50 max.	51 min.	....
0.075 mm (No. 200)	25 max.	10 max.	35 max.
Characteristics of Fraction Passing 0.425 mm (No. 40):			
Liquid Limit	....	....	b
Plasticity index	6 max.	N.P.	b
General Rating as Subgrade	Excellent to Good		

Notes: a) The placing of A-3 before A-2 is necessary in the "left to right elimination process" and does not indicate superiority of A-3 over A-2.
b) See Figure 2-23 for values.

	Silt-Clay Materials (More than 35% passing 0.075 mm)			
Group Classification	A-4	A-5	A-6	A-7
Sieve Analysis, Percent Passing:				
2.00 mm (No. 10)	....	....	....	....
0.425 mm (No. 40)	....	....	....	....
0.075 mm (No. 200)	36 min.	36 min.	36 min.	36 min.
Characteristics of Fraction passing 0.425 mm (No. 40):				
Liquid Limit	40 max.	41 min.	40 max.	41 min.
Plasticity index	10 max.	10 max.	11 min.	11 min.
General Rating as Subgrade	Fair to Poor			

Adopted from the AASHTO Standard Specifications for Transportation Materials, Part 1, p. 189, by permission of the American Association of State Highway and Transportation Officials. Year of first publication: 1973.

TABLE 2-14. UNIFIED SOIL CLASSIFICATION SYSTEM

(The No. 200 sieve size is about the smallest particle visible to naked eye)

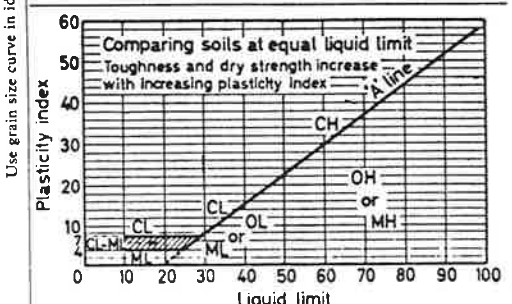
Major divisions		Sub-division	Field identification procedures (Excluding particles larger than 3 in. and basing fractions on estimated weights)	Group symbols [1]	Typical names	Information required for describing soils	Laboratory classification criteria
Coarse grained soils. More than half of material is larger than No. 200 sieve size [2]	Gravels. More than half of coarse fraction is larger than No. 4 sieve size (For visual classification, the ¼ in. size may be used as equivalent to the No. 4 sieve size)	Clean gravels (little or no fines)	Wide range in grain size and substantial amounts of all intermediate particle sizes	*GW*	Well graded gravels, gravel-sand mixtures, little or no fines	Give typical name; indicate approximate percentages of sand and gravel; maximum size; angularity, surface condition, and hardness of the coarse grains; local or geologic name and other pertinent descriptive information; and symbol in parentheses	$Cu = \frac{D_{60}}{D_{10}}$ Greater than 4; $Cc = \frac{(D_{30})^2}{D_{10} \times D_{60}}$ Between 1 and 3
			Predominantly one size or a range of sizes with some intermediate sizes missing	*GP*	Poorly graded gravels, gravel-sand mixtures, little or no fines		Not meeting all gradation requirements for *GW*
		Gravels with fines (appreciable amount of fines)	Non-plastic fines (for identification procedures see *ML* below)	*GM*	Silty gravels, poorly graded gravel-sand-silt mixtures	For undisturbed soils add information on stratification, degree of compactness, cementation, moisture conditions and drainage characteristics	Atterberg limits below 'A' line, or *PI* less than 4. Above 'A' line with *PI* between 4 and 7 are *borderline* cases requiring use of dual symbols
			Plastic fines (for identification procedures, see *CL* below)	*GC*	Clayey gravels, poorly graded gravel-sand-clay mixtures		Atterberg limits above 'A' line, with *PI* greater than 7
	Sands. More than half of coarse fraction is smaller than No. 4 sieve size	Clean sands (little or no fines)	Wide range in grain sizes and substantial amounts of all intermediate particle sizes	*SW*	Well graded sands, gravelly sands, little or no fines	*Example:* *Silty sand,* gravelly; about 20% hard, angular gravel particles ½-in. maximum size; rounded and subangular sand grains coarse to fine, about 15% non-plastic lines with low dry strength; well compacted and moist in place; alluvial sand; (*SM*)	$Cu = \frac{D_{60}}{D_{10}}$ Greater than 6; $Cc = \frac{(D_{30})^2}{D_{10} \times D_{60}}$ Between 1 and 3
			Predominantly one size or a range of sizes with some intermediate sizes missing	*SP*	Poorly graded sands, gravelly sands, little or no fines		Not meeting all gradation requirements for *SW*
		Sands with fines (appreciable amount of fines)	Non-plastic fines (for identification procedures, see *ML* below)	*SM*	Silty sands, poorly graded sand-silt mixtures		Atterberg limits below 'A' line or *PI* less than 4. Above 'A' line with *PI* between 4 and 7 are *borderline* cases requiring use of dual symbols
			Plastic fines (for identification procedures, see *CL* below)	*SC*	Clayey sands, poorly graded sand-clay mixtures		Atterberg limits shows 'A' line with *PI* greater than 7

Use grain size curve in identifying the fractions as given under field identification

Determine percentages of gravel and sand from grain size curve. Depending on percentage of fines (fraction smaller than No. 200 sieve size) coarse grained soils are classified as follows:
Less than 5% *GW, GP, SW, SP*
More than 12% *GM, GC, SM, SC*
5% to 12% *Borderline* cases requiring use of dual symbols

Identification procedures on fraction smaller than No. 40 sieve size

Major divisions	Sub-division	Dry strength (crushing characteristics)	Dilatancy (reaction to shaking)	Toughness (consistency near plastic limit)	Group symbols [1]	Typical names	Information required for describing soils
Fine grained soils. More than half of material is smaller than No. 200 sieve size	Silts and clays liquid limit less than 50	None to slight	Quick to slow	None	*ML*	Inorganic silts and very fine sands, rock flour, silty or clayey fine sands with slight plasticity	Give typical name; indicate degree and character of plasticity, amount and maximum size of coarse grains; colour in wet condition, odour if any, local or geologic name, and other pertinent descriptive information, and symbol in parentheses
		Medium to high	None to very slow	Medium	*CL*	Inorganic clays of low to medium plasticity, gravelly clays, sandy clays, silty clays, lean clays	
		Slight to medium	Slow	Slight	*OL*	Organic silts and organic silt-clays of low plasticity	
	Silts and clays liquid limit greater than 50	Slight to medium	Slow to none	Slight to medium	*MH*	Inorganic silts, micaceous or dictomaceous fine sandy or silty soils, elastic silts	For undisturbed soils add information on structure, stratification, consistency in undisturbed and remoulded states, moisture and drainage conditions
		High to very high	None	High	*CH*	Inorganic clays of high plasticity, fat clays	*Example:* *Clayey silt,* brown; slightly plastic; small percentage of fine sand; numerous vertical root holes; firm and dry in place; loess; (*ML*)
		Medium to high	None to very slow	Slight to medium	*OH*	Organic clays of medium to high plasticity	
Highly organic soils		Readily identified by colour, odour, spongy feel and frequently by fibrous texture			*Pt*	Peat and other highly organic soils	



Plasticity chart
for laboratory classification of fine grained soils

[1] *Boundary classifications:*—Soils possessing characteristics of two groups are designated by combinations of group symbols. For example *GW–GC*, well graded gravel-sand mixture with clay binder.
[2] All sieve sizes on this chart are U.S. standard.

Field identification procedure for fine grained soils or fractions

These procedures are to be performed on the minus No. 40 sieve size particles, approximately $\frac{1}{64}$ in. For field classification purposes, screening is not intended, simply remove by hand the coarse particles that interfere with the tests.

Dilatancy (Reaction to shaking):
After removing particles larger than No. 40 sieve size, prepare a pat of moist soil with a volume of about one-half cubic inch. Add enough water if necessary to make the soil soft but not sticky.
Place the pat in the open palm of one hand and shake horizontally, striking vigorously against the other hand several times. A positive reaction consists of the appearance of water on the surface of the pat which changes to a livery consistency and becomes glossy. When the sample is squeezed between the fingers, the water and gloss disappear from the surface, the pat stiffens and finally it cracks or crumbles. The rapidity of appearance of water during shaking and of its disappearance during squeezing assist in identifying the character of the fines in a soil.
Very fine clean sands give the quickest and most distinct reaction whereas a plastic clay has no reaction. Inorganic silts, such as a typical rock flour, show a moderately quick reaction.

Dry Strength (Crushing characteristics):
After removing particles larger than No. 40 sieve size, mould a pat of soil to the consistency of putty, adding water if necessary. Allow the pat to dry completely by oven, sun or air drying, and then test its strength by breaking and crumbling between the fingers. This strength is a measure of the character and quantity of the colloidal fraction contained in the soil. The dry strength increases with increasing plasticity.
High dry strength is characteristic for clays of the CH group. A typical inorganic silt possesses only very slight dry strength. Silty fine sands and silts have about the same slight dry strength, but can be distinguished by the feel when powdering the dried specimen. Fine sand feels gritty whereas a typical silt has the smooth feel of flour.

Toughness (Consistency near plastic limit)
After removing particles larger than the No. 40 sieve size, a specimen of soil about one-half inch cube in size, is moulded to the consistency of putty. If too dry, water must be added and if sticky, the specimen should be spread out in a thin layer and allowed to lose some moisture by evaporation. Then the specimen is rolled out by hand on a smooth surface or between the palms into a thread about one-eighth inch in diameter. The thread is then folded and re-rolled repeatedly. During this manipulation the moisture content is gradually reduced and the specimen stiffens, finally loses its plasticity, and crumbles when the plastic limit is reached.
After the thread crumbles, the pieces should be lumped together and a slight kneading action continued until the lump crumbles.
The tougher the thread near the plastic limit and the stiffer the lump when it finally crumbles, the more potent is the colloidal clay fraction in the soil. Weakness of the thread at the plastic limit and quick loss of coherence of the lump below the plastic limit indicate either inorganic clay of low plasticity, or materials such as kaolin-type clays and organic clays which occur below the A-line.
Highly organic clays have a very weak and spongy feel of the plastic limit.

Reproduced from "The Use of the Unified Soil Classification System by the Bureau of Reclamation," p. 126-7, by A.A. Wagner. Year of first publication: 1957.

TABLE 2-15. RECOMMENDED SUBGRADE SOIL TESTS

1. Visual-Manual Description (D2488)*

2. Material finer than No. 200 Sieve (D1140)

3. Standard Penetration Test (N) (D1586)

4. Groundwater Observation (in boreholes or observation wells)

5. Coarse-Grained Soils

   a. Grain Size Analysis (D422), (C_u, C_c, D_{10})
   b. Burmister and Unified Classifications

6. Fine-Grained Soils

   a. Moisture Content (D2216)
   b. Liquid Limit (D423)
   c. Plastic Limit and Plasticity Index (D424)
   d. Unified Classification (D2487)
   e. Unconfined Compression (D2166) or Triaxial UU (preferred)

7. Tests to be selected in a site specified testing program designed on the basis of above tests or on the basis of local experience. See text.

   a. Static Cone Penetration Test (D3441)
   b. Field Vane Test (D2573)
   c. Plate Bearing Test (D1194)
   d. Percolation Test (D3385)
   e. Shrinkage Factors (D427)
   f. Specific Gravity of Solids (D854)
   g. Organic Content (D2974)
   h. Minerologic Tests (Cation Exchange, X-Ray Diffraction)
   i. Consolidation Test (D2435) (Including measurement of swell pressure or free swell, if appropriate)
   j. Triaxial Consolidated-Drained or Undrained Tests
   k. Seismic Refraction

*Number in brackets indicates ASTM Standard if C or D prefix, or AASHTO Standard if T prefix.

In selecting soil classification and testing procedures for railroads, consideration must be given to the soils engineering technology that has been developed in other engineering areas. Of particular note are classification and testing methodologies that have been developed for highway and airfield design. Appendix B summarizes a survey made by the Transportation Research Board of methods, equipment, and boring criteria used by state transportation agencies in performing geotechnical investigations. This survey provides some perspective on the recommendations of Table 2-15.

Not all the tests listed in Table 2-15 should be performed for all projects. Rather, tests should be selected on the basis of material characteristics as these become evident. For example, visual-manual description and determination of the percent material finer than the No. 200 sieve (tests 1 and 2 of Table 2-15) serve as a basis for selecting either tests 5 for coarse-grained soils or tests 6 for fine-grained soils. The data from tests 1 through 6 will provide approximate estimates of the performance characteristics of the subgrade soils. For many railroad engineering problems, this will be sufficient for final design.

For other railroad engineering problems, tests 1 through 6 of Table 2-15 will provide insufficient data for final design. The remaining tests in the table may be desirable. These tests are not recommended for use on all projects because--

a. They are relatively expensive

b. They apply only locally

c. They do not provide a meaningful measure of important material and performance characteristics for most soils

d. There is limited experience with the test, or

e. They are not necessary.

Nevertheless, there are specific situations which the earlier tests will help identify for which some of these tests can and should be performed.

2.7 SUBGRADE PROBLEMS

The most prevalent types of subgrade problems or failures that occur in railroad track are discussed in this section. These problems range from gross foundation failures which are generally deep seated in nature (e.g., slope stability, bearing capacity, embankment settlement) to more localized concerns which are confined to action that occurs close to track elevation and are significantly affected by such environmental factors as rain, frost, and drying (e.g., mud pumping, frost heaves, erosion). In terms of the cost of maintaining the track, both types of problems are significant; however, because most track is constructed close to natural grade, more track mileage is affected more by localized subgrade deficiencies than by major foundation failures.

Major Foundation Problems

The following are descriptions of major foundation problems.

Stability - Stability problems occur when the strength of the subgrade below track elevation cannot resist the applied loads of the track and embankment. Embankment failures are the most dramatic and visible types of subgrade problems, and occur when the shear strengths of the embankment fill and the underlying subgrade and foundation are unable to resist the load of the embankment, track, and operating train. Typically, this type of failure occurs shortly after track construction or after rainfall and is characterized by a rapid drop of the embankment crest, resulting in an abrupt loss of track alignment and surface.

Stability failures also may occur after excavation. Cut slope failures generally occur after construction. Negative pore pressures in the slope and foundation dissipate over time, resulting in decreased shear strength and stability. In cut slopes, particularly in cohesive soils (i.e., silts and clays), there may be a time lag between construction of the cut and failure of the slope. This lag varies from one day to months or years, depending upon the permeability of the soil and the characteristics of the slide. Particularly troublesome are heavily overconsolidated clays, quick clays, and loose sands. Failures over time may be spontaneous or may be triggered by erosion of the toe, trimming of drainage ditches, or rainfall. A complete discussion of natural slope stability is beyond the scope of this report.

Embankment or cut slope stability is a classic soil mechanics problem that is analyzed in such textbooks as Soil Mechanics (published in 1969 by T. William Lambe and R. V. Whitman). Before evaluating short-term stability, which is critical in embankment loadings, the undrained shear strength must be determined. When examining cut slopes in which the long-term or drained condition is critical, the drain shear strength parameters must be evaluated. Progressive slope failures have sometimes occurred in cut slopes that have resulted from a combination of localized sloughing, strain softening or progressive weakening of the soil, and transfer of applied stress along the developing failure surface. This is a particular problem in stiff, overconsolidated clays. The ultimate stability of the slope may be governed by the residual shear strength, significantly lower than the peak shear strength typically measured in the laboratory.

Creep - When stability failure occurs, the applied loading of the constructed embankment or excavation creates stresses in the subgrade and foundation soils that are greater than the available peak shear strength. Such failures are normally dramatic, resulting in rapid displacement. There are some soils, however, such as soft cohesive materials that may exhibit a gradual and continuing shear strain or creep in response to an applied shear stress even though the peak shear strength of the soil is not exceeded. The rate of creep normally increases with the applied stress, and involves shear strain of the soil without a change in volume. Creep failures are characterized by limited strain that progresses slowly over time.

Consolidation Settlement - Consolidation was discussed in Section 2.2. When an embankment is constructed over soft soils, the stress on the subgrade and foundation will increase. As excess pore pressures dissipate in the subgrade, the effective stress in the soil will increase, leading to a decrease in void ratio and settlement of the embankment. In granular soil--i.e., sands, gravels, and nonplastic silts--dissipation of the excess pore pressure will be rapid, causing embankment settlement during construction. In cohesive soils, dissipation of the excess pore pressure will be relatively slow, so that embankment settlement may continue after construction of the track superstructure. Therefore, settlement generally is only significant for cohesive soils in which consolidation affects the line and surface of in-service track.

Consolidation problems also are adequately analyzed in standard soil mechanics textbooks. Soils settling under load due to dissipation of excess pore pressure may be monitored by piezometers that record the pore pressure change over time. Even after complete dissipation of excess pore pressure, soils may still continue to compress due to a phenomenon called secondary compression. Secondary compression is most significant in highly organic, highly plastic, or highly sensitive cohesive soils.

Near Surface Subgrade Problems

Problems occurring near the surface of the subgrade are presented below.

Embankment Surface Sloughs - Material may slide off the surface of the slope--even without gross slope failure--due to shallow sloughing. In soils that are susceptible to freeze-thaw action, such as silty glacial till or low-plasticity silts, a weak zone may develop in shallow depths during the thaw cycle leading to shallow slides of the surface material. For all types of soils, water flowing out from the surface of a slope can cause shallow sloughing.

Generally, surface sloughing will not immediately affect track operation. However, if not repaired, it may eventually lead to interruption of operations--e.g., in an embankment section, it may undermine the ballast shoulders and the ties, and in a cut slope, sloughing may lead to blocking of the track or drainage ditches.

Soft Subgrade Problems - The three types of soft subgrade processes confronting railroad engineers are mud pumping, ballast pockets, and squeezing. These problems are similar because they generally occur in cohesive soils with high water content. Silts and low-plasticity clays (USC classes ML, MH, and CL) are of particular concern; highly plastic clays (CH) exhibit these problems less frequently.

Mud pumping is the intrusion of subgrade soil into the ballast bed and vice versa. Here, the ballast particles are pushed into the subgrade by train traffic forces, and the soft subgrade soils are pumped up and mixed with the ballast particles around the ties. This process leads to settlement of the track and loss of drainage capability in the ballast which, in turn, decreases shear strength and resilient performance of the ballast bed. Loss of drainage can lead to rotted ties and loosened fasteners.

Ballast pockets are phenomena generally associated with jointed track. In this instance, the ballast is pushed into the subgrade at specific locations, particularly at rail joints. If the subgrade is cohesive, the permeability will be low. The depression in the ballast will then collect water, causing an increased softening of the subgrade near the depression. With repeated loading, the depression deepens and ridges of soft subgrade material collect around the pocket which contaminates the ballast and forms a larger, water-filled pocket. These ridges may be seen as "push-ups" within or at the ends of the ties. The rail dips that develop near the ballast pocket may lead to extraordinary dynamic stresses transmitted to the subgrade--which may accelerate the development of the pocket.

Squeezes are more general phenomena involving static and dynamic forces on the ballast that lead to a lateral displacement of the subgrade beyond the ends of the ties and settlement of the ballast. This condition is aggravated by excess water in the shallow subgrade either due to surface infiltration or pumping of water from below. In addition, two or three soft subgrade phenomena may be observed to occur in combination on a given section of track.

Frost-Induced Problems - Frost can cause two types of subgrade performance problems. The first is frost heave (described in Section 2.2) which develops from ice lenses that form within the soil that expand and lift the ground surface. The four conditions required to develop a significant frost heave are freezing temperatures, freezing above the water table, water at a moderate depth below the frost line, and a frost-susceptible soil. Due to the complex interaction of these four factors, the magnitude of frost heave is random, creating a rough track surface. "Shimming" beneath tie plates is the most frequent method of dealing with frost-induced track roughness; shims are removed when the weather becomes warmer.

Frost heaving and ice lens formation cause significant maintenance problems during the freezing and thawing season. However, the more prevalent frost-related concern is the loss of subgrade shear strength that occurs during thaw cycles. When subgrade thaws from the surface downward, it releases excess moisture; however, a frozen zone or ice lens may still exist which will prevent drainage of the excess moisture. If an ice lens has formed, it will turn to water, leaving behind a saturated, loosened zone. With or without actual ice lens formation, the loss of shear strength may be significant. Settlement of track may occur during the thaw season due to the previously described processes including mud pumping, squeezes, and accelerated development of ballast pockets. Shallow sloughing of embankment slopes may also be induced by freeze-thaw action, also discussed earlier.

Swelling and Shrinkage - Highly plastic soils will alter their volume when subjected to changing water content. The most significant problems are experienced by highly plastic, partially saturated, cohesive materials. Index tests for measuring swell potential are described in Section 2.3, and Table 2-4 provides laboratory and field index test parameters to qualitatively evaluate swell potential. Changes in moisture content in soils may be brought about by alterations to the surrounding environment for such reasons as the removal of vegetative cover, covering an area (e.g., with an embankment), and construction of drainage facilities through an area. Since swelling and shrinkage involves an interaction of soil stratification, soil properties, and the ambient environment, the magnitude of swell and shrinkage is nonuniform and may lead to deterioration of the track surface. The largest displacements are caused by the initial swell of partially saturated soils. Subsequent cycles of shrinkage and swelling generally lead to smaller, more uniform changes in volume and surface displacement. Swelling soils are most prevalent in the southern, southwestern, and mountain areas of the United States, and are significant contributors to track deterioration.

Collapse - As discussed in Section 2.2, some soils may be relatively stiff when loaded dry; however, they will suddenly collapse when they become saturated. If not properly identified and treated during construction, these collapsing soils can lead to progressive settlements over time. Settlement may be sudden or gradual and uneven, causing deterioration of the track surface.

Liquefaction - Vibration is an effective method of compacting cohesionless soils, particularly sands. Liquefaction--i.e., loss of shear strength due to increase of pore pressure from densification--can occur in loose, saturated, cohesionless soils (particularly coarse silts and fine to medium sands) due to earthquake loadings. Under vibratory loading, soil particles tend to become more compact, transferring stresses to pore water pressure and resulting in low to zero effective stress and a reduction in shear strength. This process may also be induced by vibratory loading from train operations. Complete or partial liquefaction of the subgrade soils beneath the track can lead to a gradual deterioration of line and surface, and may even lead to major movement of the embankment.

Erosion - Erosion must be considered in the design and maintenance of railway lines because it not only can directly affect the soundness of the track--by removing support to the ties--but it can also contribute to other types of failures. For example, a slope that initially is stable may fail if the material at the toe is eroded by water or wind. Because open drainage facilities frequently are placed adjacent to railway embankments, erosion of these embankments by drainage water or by natural water courses should always be prevented.

3. BALLAST MATERIALS FOR CONVENTIONAL RAILROAD TRACK

Ballast is any material that is spread over the subballast or subgrade to perform all of the following functions:

a. Support the track structure and maintain its alignment and grade

b. Provide a ready means for adjusting track geometry to reestablish line and grade

c. Distribute loads to underlying materials

d. Provide rapid drainage of the track and its substructure

e. Provide a resilient support layer for the track to limit transmission of dynamic wheel forces to the underlying subballast and subgrade

f. Provide an insulating layer to limit frost penetration into the subgrade

g. Provide a cover to inhibit growth of vegetation in the track.

Prior to 1970, no data were generally available on the relationship between ballast type and maintenance costs. The only cost variables that could be readily evaluated for ballast were its purchase price and hauling costs. And hauling costs were subject to interpretation because railroads often hauled the ballast on their own lines. Sources of ballast were frequently chosen by the railroads solely based on the close proximity of inexpensive, usable materials that satisfied minimum specifications set forth by the railroads. However, choosing a close source of cheaper materials often resulted in the use of materials that performed poorly and that required frequent maintenance to keep track operation safe and economical.

Two approaches have been used to study ballast. The first uses well-controlled, small-scale laboratory tests to investigate one or two performance characteristics of ballast. The second approach uses in-service track or full-scale models to systematically observe the ballast performance. It has the advantage of at least partially imposing prototype in-service loading and environmental conditions on the ballast.

Since 1970, research related to railroad ballast primarily has focused on three areas:

a. Mechanical testing of ballast to investigate deformation characteristics

b. Evaluation of ballast performance in operating track for two to five years

c. Tests of track structures up to full size with repeated loading up to one million cycles

In North America, mechanical testing of ballast has included static and cyclic triaxial tests, static and cyclic oedometer tests, and shear box tests. Research results have been published by Thompson and his colleagues at the University of Illinois in 1975 through 1978; by Raymond and his colleagues at the Canadian Institute of Guided Ground Transport (CIGGT) in 1975 through 1979; and by Dalton at the Canadian National Rail Research Center in 1977. Similar studies have been executed by various European railways, as reported by the International Union of Railways in 1970 and Dogneton, under the auspices of the International Union of Railways, in 1975.

The Association of American Railroad's Track Structures Dynamic Test Facility, the CIGGT under Raymond, and the Technical University at Aachen, the Netherlands -- sponsored by the International Union of Railways -- have studied ballast performance in model tests of single ties and multiple tie panels. In-service evaluation of ballast performance has been systematically investigated in a test section constructed by the Canadian National Railroad, as reported by Dalton in 1973(1), with further study by the CIGGT reported by Gaskin and Raymond in 1976(2). In addition, ballast performance is being investigated at the U.S. Department of Transportation's Facility for Accelerated Service Testing (FAST) track in Pueblo, Colorado.

Considerable research data that are available on the performance of earth materials as highway base courses and concrete aggregates might be appropriate for use in ballast studies, as reviewed by Robnett et al. in 1975.(3) It's important to note that railroad ballast is the most severe application of earth material aggregates in civil engineering, in terms of applied stress levels and environmental exposure.

Regardless of the approach used, in order to apply particular study results to other ballast materials or to materials in new environments, it's necessary to define the essential and quantifiable characteristics of ballast that respond to the aspects of the ballast environment and loading. In Section 3.1 essential ballast performance characteristics will be discussed. Section 3.2 will include descriptions of the index tests that may provide a measure of performance characteristics. The final section, 3.3, will present a recommended classification system and method of describing ballast materials at the source and in track.

---

(1) C.J. Dalton, "Field Durability Tests on Ballast samples as a Guide to the Significance of the Specification Requirements," Canadian National Railways Technical Research Center, St. Laurent, Quebec, Canada, January 1973, 40 pp.
(2) P.N. Gaskin and G.P. Raymond, "Contribution to Selection of Railroad Ballast," Transportation Engineering Journal, ASCE, Vol. 102, No. TE2, Proceedings Paper 12134, May 1976, pp. 377-394.
(3) Q.L. Robnett et al., Technical Data Bases Report: Ballast and Foundation Materials Research Program, U.S. Dept. of Transportation, Federal Railroad Administration, Washington, D.C., July 1975, FRA/ORD-75/138, 179 pp.

3.1 BALLAST PERFORMANCE CHARACTERISTICS

Ballast performance characteristics are divided into five categories:

a. Mechanical -- Related to the resistance of ballast to deformation and disintegration under single and repeated stresses

b. Environmental -- Related to the resistance of ballast to alteration due to changes in temperature, water, or other nonmechanical factors

c. Permeability -- Related to the passage of liquid (e.g., water) and solids (e.g., fine particles) through the ballast

d. Electrical -- Related to electrical conductivity or resistivity of the ballast

e. Construction (Maintenance) -- Related to the ease with which tamping, lining, and other operations may be carried out on the track.

In the following sections, each class of characteristics will be discussed as it may relate to the ballast functions described in the introduction to this section.

Mechanical Characteristics

Mechanical characteristics relate to the following ballast functions:

a. Limit vertical, lateral, and longitudinal tie movements

b. Reduce static subgrade stresses due to wheel loads

c. Dampen dynamic overstress.

These functions relate to the static, resilient, and permanent deformation characteristics of ballast. The following mechanisms will affect deformations or strains within the ballast section:

a. Elastic and inelastic shear strain of particles

b. Rearrangement or densification of particles

c. Disintegration or particle breakdown into smaller sizes

d. Cementing of the coarse ballast particles with fine contaminants into a concrete-like mass.

Robnett et al. in 1975(1) and Selig et al. in 1979(2) provide in-depth reviews of mechanical behavior of ballast. Research into laboratory behavior of ballast materials and some field and model tests have led to the following general conclusions about mechanical characteristics of ballast and most other granular materials:

a. Peak shear strength and resilient modulus or stiffness of the materials increase significantly with an increase in the initial bulk density of ballast; i.e., the smaller the initial void ratio, the higher the strength and stiffness.

b. Strength and stiffness of the materials also increase significantly with an increase in the level of confining stress.

c. There is an uncertain relationship between maximum particle size and mechanical properties. Some studies report an increase in strength and stiffness in proportion to an increase in particle size, whereas other studies report no correlation between mechanical properties and particle size. Broader grading (i.e., a wider range of particle sizes) may decrease the elastic modulus while increasing ballast resistance to inelastic strain and ultimate strength.

d. Increased particle angularity produces an increase in the dilatancy of the ballast, i.e., the ballast tries to expand when sheared. Dilatancy leads to increased stiffness and shear strength.

e. Increased saturation significantly lowers strength and stiffness. Since the testing described in the research reports was done under drained conditions, this effect is not attributed to pore-pressure buildup.

f. The magnitude of permanent strain resulting from cyclic loading is approximately proportional to the logarithm of the number of applied load cycles. That is, the permanent strain per cycle decreases as the number of applied cycles increases. The strain during the first applied load cycle is roughly equal to the strain that accumulates during the succeeding 100,000 cycles.

g. Permanent volumetric strain of ballast is a result of both reorientation of particles to a denser state and disintegration of particles into smaller pieces by abrasion and fracture.

---

(1) Q.L. Robnett et al., Technical Data Bases Report.
(2) E.T. Selig, T-S Yoo, and C.M. Panuccio, Mechanics of Ballast Compaction: Technical Review of Ballast Compaction and Related Topics, Vol. 1, prepared for USDOT-TSC, March 1982, Report No. FRA-ORD-81-16.1, 287 pp.

Environmental Characteristics

Because ballast is placed at the surface of the track section, it's subjected to temperature and humidity extremes of the natural environment. Ballast is also periodically exposed to chemicals carried by rain water, flood water, and spillage from cars. The following attributes of ballast directly affect its function.

Freeze-Thaw Resistance - Except in the extreme South, railroad trackage in North America is subjected to subfreezing temperatures each year. Calculation of the thermal regime of a track section in Illinois, as reported by Knutson et al. in 1977, indicates that the entire thickness of the ballast section in northern latitudes will be subjected to at least several freeze-thaw cycles annually(1). Therefore, freeze-thaw resistance is an important ballast characteristic.

Freeze-thaw degradation is probably caused by swelling pressure of water freezing in the confined pores of rock. The degree of water saturation of the pore space is probably the most significant factor influencing frost action because the potential swelling pressure caused by freezing water in a confined space can split most rocks. Therefore, freeze-thaw damage potential might be determined by evaluating the degree to which ballast pores can be filled with water under natural conditions.

Freeze-thaw actions may affect ballast performance two ways:

a. The pressures caused by freeze-thaw within the water-filled voids of the particles may induce disintegration

b. Freeze-thaw may reduce the elastic and inelastic mechanical resistance of the ballast.

Wet-Dry Resistance - Some rock materials, such as clay stones, mud stones, and some shales, will disintegrate if subjected to alternate cycles of wetting and drying without confinement. Such materials are weak, soft, and generally unacceptable for railroad ballast. If a ballast material is adversely affected by wet-dry action, it will also be severely deficient in such other characteristics as abrasion, strength, and freeze-thaw resistance. Therefore, wet-dry resistance of proposed ballast materials is ordinarily insignificant compared with more stringent performance characteristics. However, sound, hard ballast may be contaminated with friable particles or clay lumps which are susceptible to break-down under wet-dry action.

---

(1) R.M. Knutson et al., Ballast and Foundation Materials Research Program, Materials Evaluation Study, U.S. Dept. of Transportation, Federal Railroad Administration, Washington, D.C., FRA/ORD-77/02, 324 pp.

Chemical Activity Potential - The chemical alteration or weathering of rocks has long been discussed in geology literature(1), and investigated by highway engineers. Although recognized as a significant factor in ballast performance, systematic studies of chemical alteration have only been recently undertaken, as reported by Raymond in 1977(2).

All geologic materials weather in time. However, the rate of chemical alteration varies and is a function of many factors, including rock mineralogy, temperature, available moisture, chemical contaminants, particle surface area, and particle size. Except in unusual cases, it's not possible to anticipate the environment to which ballast will be exposed. Using the knowledge developed in the fields of geochemistry and petrography, however, it may be possible to evaluate the relative potential for chemical alteration of various ballast sources.

Permeability Characteristics

Here, permeability refers to the ballast characteristic that influences the flow rate of water and the related characteristic controlling the migration of fine particles through the ballast. Generally, it's acknowledged that (1) soil permeability and migration of fines in soil are both related to the size of the pores or voids in a granular medium, and (2) the pore size is related to average particle size and gradation and the degree of consolidation or in-place density.

Fresh ballast that satisfies specifications for rock, slag, or gravel ballast used by most railroads normally will exhibit high permeabilility to water -- at least 1 mm/sec -- whereas the permeability of natural soils other than clean gravels will be lower. Therefore, fresh ballast constructed in a normal cross section will provide for prompt initial drainage of the ties.

In track, however, the permeability of ballast typically deteriorates with time due to fouling and compaction caused by service loads. Fouling is the contamination of ballast with fine-grained material. Internal contamination may be due to disintegration of ballast particles caused by mechanical disintegration, freeze-thaw degradation, chemical weathering, or unsuitable contaminants originally delivered with the ballast. External contamination may result from (1) introduction of fine material that is blown or washed into the top or side of the ballast section, dropped on the ballast during locomotive sanding operations, or spilled from leaky hopper cars; (2) rail dust; or (3) material from beneath the ballast that intermingles with the ballast (due to pumping). When the voids in the ballast become fouled or clogged with fine material, ballast permeabililty will then be contingent upon the permeability of the fine contaminants.

---

(1) R.E. Aufmuth, Site Engineering Indexing of Rock, ASTM Technical Publication No. STP554, 1974, pp. 81-99.
(2) G.P. Raymond, Stresses and Deformations in Railway Track, prepared for the Canadian Institute of Guided Ground Trasport, CIGGT Report No. 77-15, Queen's University, Kingston, Ontario, Canada, November 1977.

The most significant factor causing internal ballast contamination is the tendency of the fine materials in ballast to form plastic or clayey fines which have very low permeabilities. Some fine particles may in time form a cement, binding the coarse ballast particles into a rigid mass.

The ease of particle migration within the ballast is intimately related to the problem of external fouling. If the ballast is composed of coarse particles that are a uniform size, the ballast will contain large voids that permit rapid migration of fine particles to the bottom of the ballast section where they collect and form a relatively low permeability layer. In the worst case, this layer may eventually cause water to pond in the ballast layer, thereby saturating the ballast. As discussed previously, saturation significantly impairs mechanical performance. Saturation may also accelerate chemical alteration, freeze-thaw degradation of the ballast, and rotting or other deterioration of the ties and loosening of fasteners.

Railroad engineers currently favor ballast with a high percentage of coarse particles from 19 mm up to as much as 65 mm and large open voids for the following reasons:

a. The open voids promote migration of fines to the bottom of the ballast section where there is considerable void space. When desired, the layer of fouled ballast at the base of the ballast section can be separated from the ties by lifting the track and adding fresh ballast to the top of the section.

b. The abundance of coarse particles facilitates maintenance with current lining and tamping equipment.

Some ballast research tests, however, indicate that more broadly graded ballast (i.e., a wide range of particle sizes) tends to resist permanent deformation better than uniformly graded (homogeneous particle size) materials. Broadly graded materials might be cheaper to produce since they require culling a smaller fraction of the undersized product coming from the crusher. Finally, the more broadly graded materials may tend to limit the migration of fines downward from the surface to the bottom of the ballast section or from the subgrade upward, thereby slowing the rate at which ballast permeability falls to an unacceptable level. It's still unclear how best to grade ballast to optimize permeability, migration, resistance to permanent strain, and ease of maintenance because ballast gradation has generally received little attention in ballast research studies on in-service track.

Consolidation or densification of ballast will also cause a decrease in void size and a decrease in permeability. However, the change in permeability due to consolidation is a second-order effect compared with fouling, and therefore warrants little attention at this time.

Electrical Characteristics

High electrical resistance of ballast is important in order to prevent interference with control signals. Track rails are used as conductors to operate signals; if ballast causes a short circuit path from the rails to the ground or between the rails, the signal path will be completed and the signal will trip.

However, electrical conductivity normally is not a significant consideration for ballast materials. It may prove important for high-metallic sources such as some iron ores or slags from metal processing, aggregates with a high sulphur content, materials that form alkaline or chloride salts, and coral. In 1962, the American Railway Engineering Association (AREA) reported on the conductivity of iron furnace slags; they concluded that wet slag materials are much more conductive than dry slag, but there is no significant difference in conductivity of wet ballast among the various materials tested.(1) Electrical conductivity generally will not limit the use of proposed ballast materials. Therefore, electrical characteristics will not be considered in detail in this report.

Maintenance Characteristics

Lining and surfacing operations by machine or manual tamping may impart an impact and abrasive loading condition on ballast that is more severe than any other mechanical action. The power tamper vibrating blade which acts directly on the ballast tends to fracture and abrade the particles. Tamping with manual picks produces even greater fracture-inducing effects.

Tamping forces ballast particles under the ties to support the track at the proper level. In the process, the tamping tool must push coarse ballast particles from the cribs between the ties to beneath the ties. To operate this way, a high percentage of coarse particles, generally in the 25-mm to 50-mm range, is desirable. However, if the particles are too coarse, tamping may result in the ties resting on the corners of only a few large pieces. Subsequent service loading will induce severe initial settlement when the relatively few particles are overloaded.

---

(1) The Signal Manual of the Communication and Signal Section of the Association of American Railroads recommends a minimum dry resistance of ballast of 3 to 5 ohms per 1000 feet; wet ballast should exceed 0.25 ohms per 1000 feet. The AAR Signaling Principles and Practices Manual, Chapters VII, XI, and XXV indicate procedures for measuring ballast resistance in the field.

3.2 BALLAST INDEX TESTS

The following list of material properties that influence ballast performance were derived from characteristics discussed previously. Each performance characteristic heads a list of material properties that affect the characteristic.

a. Mechanical Performance
   - Physical state or ballast density
   - Particle shape
   - Hardness and toughness
   - Saturation/water content
   - Confining pressure
   - Cementing potential of fines
   - Gradation/particle size

b. Environmental Performance
   - Freeze-thaw resistance
   - Chemical activity of minerals
   - Clay lump, friable and soft particle content
   - Petrographic features

c. Permeability Performance
   - Gradation
   - Fines permeability

d. Electrical Performance
   - Electrical resistance

e. Maintenance Performance
   - Hardness and toughness
   - Gradation

A collection of index tests and engineering property tests that may be used to measure the material properties are listed in Table 3-1. The American Society for Testing and Materials (ASTM), or the American Association of State Highway and Transportation Officials (AASHTO), or other published procedures are referred to for each test. Because of the large size of ballast particles, many of the referenced procedures must be modified to be suitable for ballast.

The state-of-the-art in ballast evaluation currently provides minimal guidance for selecting those tests that relate to field performance. The current data on ballast performance and index test correlations are insufficient to rule out any test as a promising indicator of ballast performance. For practical reasons, the number of index tests used to characterize ballast materials should be limited.

Several of the tests listed in Table 3-1 indicate the same material property of the ballast. The attributes of these comparable tests are discussed in this section. The factors evaluated will be (1) the ability of the test to correlate with a particular performance characteristic; (2) precision, ability to repeat, and accuracy of the test; and (3) effort involved or cost of running the test.

TABLE 3-1. INDEX TESTS FOR BALLAST

Mechanical Characteristics

Physical State

1. Reference Density [Selig, 1977]*
2. Relative Density [D2049]
3. Void Ratio (Porosity) [C29, C30]
4. In Situ Density [D2167, Selig, 1977]
5. Particle Index Test [D3398]
6. Proctor Compaction Test [D698, D1557]

Particle Characteristics
(Shape, Angularity, Surface Texture)

1. Sphericity [Pettijohn, 1957]
2. Angularity [Pettijohn, 1957]
3. Flakiness Index [BS812]**
4. Elongation Index [BS812]
5. Particle Index Test [D3398]

Hardness and Toughness

1. Los Angeles Abrasion Test (dry, wet) [C131, C535]
2. Deval Attrition Test (dry, wet) [D2 (deleted), T3]
3. Production of Plastic Fines (California Durability) [T210]
4. Crushing Value [BS812]
5. Scratch Hardness [C235]
6. Abrasion Resistance [C241]
7. Point Load Strength [I.S.R.M., 1974]
8. Unconfined Compressive Strength [D2938]
9. Brazilian Tensile Test [C496]
10. Page Impact Hardness [BS812]
11. Dorry Abrasion [BS812]
12. Toughness (Impact Strength) [D3 (deleted)]
13. Shore Scleroscope
14. Schmidt Hammer Hardness
15. Aggregate Impact Resistance [BS812]
16. Mill Abrasion [Raymond, 1979]
17. Clay Lumps, Friable Particles [C142]
18. Petrographic Analysis (Intact Particles and Fines) [C295]

Saturation and Water Content

1. Natural Water Content [D2296, C566]
2. Void Ratio, Porosity [C29, C30]

Confining Pressure

1. In Situ Density [Selig, 1977]
2. Reference Density [Selig, 1977]
3. Bulk Specific Gravity [C127]
4. Grain Density [I.S.R.M., 1972]

---

* Numbers in brackets indicate ASTM standard if C or D prefix, AASHTO standard if T prefix, or refer to reference in bibliography if name and year. ASTM is preferred over AASHTO preferred over other reference.

** British Standards Institute Standard Tests, vol. 812.

TABLE 3-1. INDEX TESTS FOR BALLAST(Continued)

Cementing Potential of Fines
1. Cementing Value Test [Nichols, 1977]
2. Petrographic Analysis of Fines [C295]
3. Hydrometer Grain Size Analysis of Los Angeles Abrasion Fines [C535,D422]
4. Atterberg Limits of Los Angeles Abrasion Fines [C535, D423, D424]

Environmental Characteristics

Freeze-Thaw Resistance
1. Freeze-Thaw Soundness [T103]
2. Sulphate Solution Soundness (magnesium, sodium) [C88]
3. Water Absorption [C127]
4. Porosity, Effective Porosity [C127]

Chemical Activity of Minerals
1. Petrographic Analysis [C295]
2. Carbonate Solubility [D3042]
3. Los Angeles Abrasion (dry, wet) [C131, C535]
4. Deval Attrition (dry, wet) [T3]

Clay Lumps and Soft, Friable Particles
1. Clay Lumps, Friable Particles [C142]
2. Scratch Hardness [C235]
3. Los Angeles Abrasion [C131, C535]
4. Deval Attrition [T3]
5. Production of Plastic Fines (California Durability) [T210]

Petrographic Features
1. Petrographic Analysis [C295]
2. X-ray Diffraction Analysis
3. Differential Thermal Analysis

Permeability Characteristics

1. Sieve Analysis [C117, C136, D422]
   - Fineness Modulus [C125]
   - Talbot n-factor [Talbot, 1923]
   - D_{10}, Coefficient of Uniformity and Curvature [Casagrande, 1948]
2. Oedometer/Permeability Test of Los Angeles Fines [raymond, 1977]
3. Petrographic Analysis [C295]

Electrical Characteristics
1. Bulk Conductivity

Maintenance Characteristics
1. Sieve Analysis (see Permeability above)
2. Abrasion and Fracture Resistance (see Hardness and Toughness above)

Engineering Properties
1. Triaxial Test (static, cyclic) [Knutson, 1977]
2. Oedometer Test (static, cyclic) [Raymond, 1977]
3. Direct Shear Test [Dalton, 1977]
4. Sonic Velocity [D2845]

Physical State - In a review of previous work related to ballast density measures in 1977, Selig et al.(1) compared the merits of the various tests that might be used to establish standard reference densities for ballast materials. Reference density is a material physical state which can be produced by a standard test procedure that can be consistently repeated. The standard Proctor test (ASTM D698) is an example of a laboratory test that provides a reference density for materials.

Several methods were compared in laboratory experiments. The results of the study indicate that both impact and vibratory compaction methods are suitable to develop reference density for ballast. However, vibratory compaction requires more elaborate equipment, and the mechanisms and energy of vibratory compaction are poorly understood. Frequency, amplitude, and duration of vibration are known to influence density achieved, and their effects are dependent on characteristics of the ballast material.

For impact compaction, metal-end impact tools should be avoided on ballast because they tend to disintegrate as well as densify the particles. A rubber-tipped impact tool similar to a 10-pound Proctor hammer was adapted by Selig for use in a 0.30-m diameter by 0.30-m high mold. Several different compactive efforts were recommended to produce a range of reference densities.

Bulk density of an aggregate is a function of two factors: particle density and interparticle void content or porosity. Because the tests in this category are intended to show physical state, results should be expressed in terms of void ratio or porosity in addition to bulk density to eliminate the variable of particle density.

Particle Characteristics - Direct caliper measurement of particle dimensions is the most fundamental of the particle shape tests, however, the test has two drawbacks. First, it's not clear which dimensions should be used for an irregularly shaped particle. Making the measurements requires a level of skill and subjective judgment. Second, development of a shape index requires individual handling of a large number of particles. This is a slow, tedious, and costly procedure.

Particle angularity is most frequently described by subjective comparison of particle silhouettes to standard silhouettes of particles. Individual examination of many particles is also required for this index measurement.

---

(1) E.T. Selig, T-S Yoo, and C.M. Panuccio, Mechanics of Ballast Compaction: Field Methods for Ballast Physical State Measurements, March 1982, Report No. FRA-ORD-81-16.2, 155 pp.

Determination of the flakiness and elongation indices by the British Standards Institute procedures(1) is simpler. Flaky and elongated particles are defined within certain limits of length ratios. Using special slotted sieves, the flakiness test is relatively simple and fast to administer, although manipulation of the pieces on the sieves may be necessary to ensure accurate results. Elongation requires a similar manual comparison of individual particle sizes to a fixed length gauge.

It would be desirable to have a test that would indicate particle shape and angularity without requiring manipulation of the individual particles. In 1962, Huang(2) proposed a particle index test that uses the changes in void volume with increases in compaction as a measure of particle shape, angularity, and surface texture. The test requires several fillings of a standardized compaction mold using different compactive energy. Similarly, in 1966, Hughes and Bahramian(3) proposed a test that involved filling a 0.15-m by 0.30-m mold in a loose state. The reference density test proposed by Selig et al. in 1982(4) involves a procedure similar to the particle index test. If the reference densities are converted to void ratio, the reference density test may serve the added function of providing an indirect measure of particle characteristics.

Hardness and Toughness

Hardness -- or resistance to abrasion -- is determined by the ability of a mineral grain to scratch another grain or be scratched by another grain. Toughness describes resistance to impact and bond strength that holds the individual grains together.

Because hardness and toughness are distinct properties, two index tests should be used to measure these two factors. One might choose a test that measures each property individually. Alternatively, a test measuring a combination of properties could be paired with a test influenced by either hardness or toughness.

---

(1) British Standards Institute, Methods for Sampling and Testing of Mineral Aggregates, Sands and Fillers, Vol. 812, Part 1: "Sampling, Size, Shape, and Classification," ISBN: 0 580 087441, 1975, 24 pp.; Part 2: "Physical Properties," ISBN: 0 580 08754 9, 1975, 20 pp.
(2) E.Y. Huang, "A Test for Evaluating the Geometric Characteristics of Coarse Aggregate Particles," ASTM Proceedings, Vol. 62, 1962.
(3) B.P. Hughes and B. Bahramian, "Laboratory Test for Determining the Angularity of Aggregate," Magazine of Concrete Research, No. 18, September 1966, pp. 147-152.
(4) E.T. Selig et al., Mechanics of Ballast Compaction.

The Los Angeles Abrasion (LAA), Deval Attrition, and production of plastic fines tests impose abrasive and impact loadings on the particles. The Los Angeles test is most frequently used and often referred to in ballast specifications in North America, such as by the AREA and the Canadian National Railway. It's currently preferred in this continent to the Deval test because the results of the former test are more consistently repeated. Some of the other tests shown in Table 3-1 are used to measure either hardness or toughness, while limiting the influence of the alternate property. Petrographic analysis attempts to estimate hardness and toughness by a visual examination and evaluation of the mineral constituents, grain size, grain bonding, and other properties that are related to hardness and toughness.

Hardness is a property that can only be correctly applied to single mineral particles. Since most rocks comprise many different minerals, it's uncertain which mineral hardness value to ascribe to a multi-mineral rock. The scratch hardness and ASTM abrasion resistance tests are limited because they involve scratching only a few grains of a heterogeneous rock. The Dorry abrasion and mill abrasion tests apply a general surface abrasion on rock particles to produce fines. The quantity of fines produced is used as an index of hardness.

In the Dorry test, rock pieces are weighted against a rotating steel plate on which quartz sand is gradually released; the abrasion resistance index is determined by the weight of material removed by this standard process. In the mill abrasion test, as suggested by Raymond in 1979(1), ballast particles are placed with water in a circular porcelain mill pot. The pot is then rotated a prescribed number of times, and the fraction of material produced in the pot that is finer than a No. 200 sieve (0.075 mm) is used as an index of the abrasion resistance of the ballast. Raymond notes that it's important to determine abrasion resistance because the very fine abraded particles produced in this test may cause a severe reduction in ballast permeability and cementing of the ballast layer.

Toughness is a more natural material characteristic for rocks because it relates to the bonding of the individual grains to each other regardless of mineralogy. In the crushing tests, a quasi-static pressure of 10 to 20 MPa (1450 to 2900 psi) is applied to a sample contained within a steel cylindrical mold. In the impact tests, a ram is dropped onto the top of the sample held within a steel mold. The range of impact energies ranges from the British Rail test that uses a 14 kg ram dropping 38 cm for 15 blows to the Austrian test that applies 20 blows of a 50kg ram dropping 2 m. In all cases, the increase in the fine particle fraction is used as an index of aggregate toughness or durability. These tests are more fully described in the 1967 report entitled, "Classification of Ballast Prescriptions," by the International Union of Railways(2).

---

(1) G.P. Raymond, "Ballast Properties that Affect Ballast Performance," American Railway Engineering Association Bulletin, Vol. 80, No. 673, June-July 1979, pp 428-449.
(2) International Union of Railways, Classification of Ballast Prescriptions, Interim Report D71/RP 7/E, June 1967, 62 pp.

The unconfined compressive and Brazilian tensile tests are frequently used in rock engineering practice. However, these tests require precisely trimmed cylindrical specimens for testing. The point load strength test uses pointed platens to load rock pieces so that irregular ballast particles, cylindrical cores, or blocks can be tested. An empirical correction factor is used with this test to adjust for the influence of particle size. The point load strength test can be completed quickly, using either standard hydraulic compression test equipment or a light, portable, and special-purpose apparatus. The remaining tests measure the rebound of an impact point to determine toughness. Most were originally developed for gauging the strength of concrete, but could be applied to ballast.

Confining Pressure

The confining pressure of the ballast is dependent on the thickness of the ballast above the base of the ties and the in-situ density of the ballast. The most significant test related to confining pressure is a measurement of in-situ density.

In 1979 Selig et al. reviewed the several methods of measuring ballast in-situ density and determining a ballast reference density in their report, Technical Review of Ballast Compaction and Related Topics. Two basic methods of in-situ density measurement are available - nuclear methods and weight-displacement methods. In nuclear density meters, the attenuation of gamma radiation is used as an index of mass density. Typically, the gamma source is placed beneath the surface of the material layer and the gamma radiation is measured by a detector at the surface. For ballast, the principal difficulty in this method is to install the gamma source within the ballast. It is impossible to drive the gamma source probe into ballast without disturbing the material so much as to make the measurement meaningless. The source could be inserted in a tube that remains beneath the surface of the ballast permanently. But this limits measurements to a few discrete locations where the tubes are installed during track construction. This option may be practical for limited research needs.

In the weight-displacement methods, ballast is excavated beneath the top surface of the ballast layer. The weight of this material is determined. The volume of the excavated hole is measured by weighing the amount of material of known density required to refill the hole. Both sand and water are used for in-situ density measurements of soils. For ballast, however, the large void sizes would lead to loss of sand or water to outside of the volume of the excavated hole. Thus, a membrane liner is required to contain the volume measuring medium. In their 1982 report on Field Methods for Ballast Physical State Measurements, Selig et al. decided on using water contained by a flexible rubber membrane as a practical and satisfactory means to determine the volume of the excavated ballast. In-situ density is calculated by dividing the weight of ballast removed by the volume of the hole.

Cementing Potential of Fines

The cementing value test measures the unconfined compressive strength of cubic samples molded from ballast fines that are produced in a ball mill pot or Los Angeles abrasion drum. The test was used by several railroads in developing their ballast specifications; however, AREA does not endorse the test because it's uncertain whether test results can be repeated consistently.

Petrographic analysis, hydrometer grain-size analysis, and Atterberg limits tests of ballast fines have been proposed to evaluate cementing potential. Based on field data, there has been no correlation between cementing potential and these indices. None of the proposed tests for cementing potential includes any account of chemical alteration of minerals from the weathering of fines.

Freeze-Thaw Resistance

The freeze-thaw soundness test attempts to model in the laboratory the freeze-thaw action that takes place in the field. The test is not widely recommended for several reasons. First, it requires special, costly freezing equipment. Second, it requires considerable time to complete. Finally, test results are influenced by a large number of factors including freezing rate and temperature, thawing temperature, the drying-wetting procedure prior to freezing, and quality of water. Difficulty in standardizing the test has led the ASTM to omit the procedure from its current specifications. It is included, however, in the AASHTO standard tests.

Sulfate soundness tests are the most commonly used index tests for measuring freeze-thaw resistance. The test imitates the action of freezing water by its formation of salt crystals that are produced in the pores of the rock by repeated soaking of samples in saturated salt solutions and oven drying. In highway and concrete technologies, magnesium sulfate is preferred to sodium sulfate because the solubility of the magnesium salt is less sensitive to test temperature than sodium salt. Sodium sulfate is most often mentioned for use as ballast material specification test, as by the AREA. Inter-laboratory test result correlations, reported by West et al. in 1970(1), are more consistent when using the magnesium sulfate test. Results of the two different tests are not comparable. Both soundness tests involve wet-dry and hot-cool cycling in addition to crystal formation. Generally, five soak-dry cycles are used in testing; however, Nichols presented data in 1977(2) to suggest that comparisons at 10 or 20 cycles may be more pertinent for determining ballast quality.

---

(1) T.R. West et al., Tests for Evaluating Degradation of Base Course Aggregates, National Cooperative Highway Research Program, Report No. 98, 1970, 92 pp.
(2) F.P. Nichols, Jr., Toward Better Evaluation of Railroad Ballast Aggregate, research report prepared for the National Crushed Stone Association, April 1977, 4 pp.

Freeze thaw resistance is critically dependent on the capacity of the rock pores to become saturated with water. A partially saturated rock will be able to withstand freeze-thaw far better than a rock with no air-filled voids. Water absorption, total porosity, and effective porosity are inexpensive measures of the saturation capability of rock, and may provide an indirect measure of freeze-thaw resistance.

Chemical Activity of Minerals

No ballast specifications in North America consider the chemical alteration potential of ballast per se. Some specifications preclude carbonate rocks (limestone) entirely. The European ballast specifications that consider chemical alteration use petrographic analysis -- most often mentioned as the best way to identify rock minerals that are susceptible to chemical alteration in a natural environment.

A trained petrographer will identify minerals in a rock, estimate percentage composition, and evaluate the chemical activity of the minerals. The analysis may use a hand lens examination of rock samples, thin section analysis, polished section analysis, or examination of ground rock particles. X-ray diffraction, differential thermal analysis, and even electron microscopy may also be used. The evaluation is mostly subjective, and the value of the analysis will obviously be dependent on the skill and experience of the petrographer.

Carbonate solubility may be a useful index test for limestones to help assess the potential for solution in service. Solubility could be determined by measuring the percentage solid weight loss due to immersing a standard gradation ballast sample in a standard acid solution for a specified period of time and temperature. It may also indicate the relative proportion of calcite and dolomite in the limestone. (Dolomite is less soluble and harder than calcite.) There are no available data relating the results of the carbonate solubility test to field weathering potential.

The Deval abrasion or attrition test -- both dry and wet -- has been used by the British and French railroads to test ballast materials, as reported by Jenkins in 1976(1) and Paterson in 1972(2). The British note that the wet attrition test is the principal index property test in which limestones are found to be deficient. West et al. reported in 1970(3) that some basaltic rocks showed markedly increased losses when water was introduced in the LAA drum. Therefore, an increase of wet abrasion test loss over dry abrasion test loss may indicate chemical activity of the rock-forming minerals.

---

(1) H.H. Jenkins, "Track Maintenance for High-Speed Trains," American Railway Engineering Association - Bulletin, Vol. 77, No. 658, June-July 1976, pp. 499-521.
(2) A. Paterson, "Matching the Track to the Load," Railway Gazette International, Vol. 128, No. 2, February 1972, pp. 53--56.
(3) T.R. West et al., Tests for Evaluating Degradation of Base Course Aggregates, NCHRP Report 98, 1970, 92 pp.

Ordinarily, the Los Angeles drum isn't designed to be water tight. Minor modifications to the standard door may be required before performing the LAA test with water.

Clay Lumps and Soft, Friable Particles

Section 2.4 of the AREA *Manual* prescribes maximum limits for clay lumps and soft particles. The relevant ASTM tests are also referred to in the manual. Both tests require manipulation of individual particles.

Several abrasion tests will demonstrate the influence of clay and soft particles because they will increase the amount of losses caused by abrasion. It may be possible to distinguish the breakdown of unusually soft contaminants by measuring the amount of loss occurring after a small number of cycles and the normal number of cycles called for in the standard test procedure.

Petrographic Features

Petrographic analysis -- described previously under "Chemical Activity of Minerals" -- is frequently used in evaluating concrete and highway base course aggregates. Petrographic analysis is included as part of several European railroad ballast specifications.

Petrography is that branch of geology dealing with the description and systematic classification of rocks. This science includes descriptions of rock texture, mineral associations, and chemical compositions of rocks. In addition to accurately classifying ballast, petrographic analysis may be used to qualitatively determine the following physical characteristics of earth materials:

a. Hardness--from mineralogic identification

b. Toughness--from evaluation of grain size, grain boundaries, grain orientation, cement composition, degree of induration, and examination for chemically altered minerals

c. Freeze-thaw resistance--from evaluation of pores

d. Weathering tendency--from examination for easily altered minerals

e. Permeability--from analysis of attrition test fines. Raymond et al. suggested in 1976(1) that angular fragments produce high permeability fines, whereas powdery fines are relatively impermeable and may be easily changed to clays.

---

(1) G.P. Raymond, R.W. Lake, and C.J. Boon, *Stresses and Deformations in Railway Track*, final report for the Canadian Institute of Guided Ground Transport, CIGGT Report No. 76-11, November 1976, 171 pp.

The ASTM-recommended practice for petrographic evaluation (C295) is intended for use in examining concrete aggregates. Petrographic examination of railroad ballast should follow those practices that are directed toward providing information that is pertinent to ballast performance, as outlined by Raymond in 1979 in "Ballast Properties that Affect Ballast Performance."

Permeability

In direct investigations of ballast permeability, Raymond et al. performed oedometer-permeability tests on the minus No. 40 sieve fines produced in 1,000 cycles of the Los Angeles abrasion device. The relationships among vertical pressure, void ratio, and permeability were investigated directly. The results of this testing were compared with the Canadian National Railroad field tests reported by Dalton in Field Durability Tests on Ballast Samples. A high fines permeability (>0.0003 cm/sec) generally correlated with good field drainage performance and with materials for which the fines were nonplastic. These high permeability materials were generally made up of fines with hard, angular fragments produced by fracture, rather than a rock flour produced by abrasion of soft minerals. Therefore, the petrographic analysis of crushed fines might be used to qualitatively assess permeability of ballast fines as well as hardness and toughness of the intact rock. In addition, those minerals that are likely to evolve into clay minerals leading to very low permeability may be identified.

Permeability is frequently related to grain size. Sieve analysis is the most frequently used test to measure grain sizes of coarse particles. A graphic display of weight (in percent) that passes through each sieve versus the logarithm of the sieve opening size is the usual means of reporting test data, as shown in Figure 3-1. However, it's desirable to express grain size characteristics of material (e.g., maximum size, average size, and size distribution) by one or two numerical parameters.

There are four parameters listed in Table 3-1, under "Permeability Characteristics," that have been used previously to describe grain size characteristics. The fineness modulus is an empirical factor obtained by adding the percentages of a sample of aggregate retained on each of a specified series of sieves and dividing the sum by 100(1). The fineness modulus is used most frequently to characterize fine aggregate in concrete mix design. The calculation is simple; however, a standard set of sieves must be used in the test. Material gradation finer than the No. 100 sieve is ignored. The fineness modulus for a broadly graded ballast gradation curve is shown in Figure 3-1.

---

(1) Sieve sizes: U.S. Standard No. 100, No. 50, No. 30, No. 16, No. 8, No. 4, 3/8 inch, 3/4 inch, etc.; openings increasing by a factor of 2.

The n-factor proposed by Talbot and Richart in 1923(1) represents the gradation curve by the mathematical function

$$p=(d/D_m)^n \times 100$$

where

p = the percentage of material finer than size d

d and D_m = the individual sieve size and the maximum sieve size of the material respectively

n = the empirical parameter that is a measure of the broadness of the particle size gradation.

Thus, the gradation of a particular material may be represented by the two parameters, D_m and n. This method of describing gradation may be used with any set of sieves; however, a curve-fitting procedure must be used to determine the parameters D_m and n. Unusual or gap-graded size distributions are not well represented. A plot of gradation curves described by the Talbot equation are also shown in Figure 3-1.

In 1948, Casagrande(2) stated that the significant index property of coarse-grained soils was grain size. As described in Chapter 2, grain diameters larger than 10, 30, and 60 weight percent of the total sample (D_{10}, D_{30}, D_{60}) were recommended to describe grain size distribution. Two ratios were defined: the coefficient of uniformity--$C_u=D_{60}/D_{10}$--and the coefficient of curvature--$C_c=(D_{30})^2/(D_{10} \times D_{60})$. Thus, three parameters may be used to describe the gradation of a coarse-grained soil. The parameters chosen by Casagrande concentrated on the finer portions of the sample that may most significantly influence the performance characteristics of sand and gravel. However, such a description minimizes the significance of the coarsest material that may be relatviely more important for ballast behavior.

In 1969, Hudson and Waller(3) developed a gradation modulus ($\overline{A}$) for studying aggregate gradation effect on concrete strength. The gradation modulus for a particular size fraction -- i.e., the material contained between two sieves -- is

$$\overline{A}_i = \log_2 (54.8/\overline{d}_i)$$

---

(1) A.N. Talbot and F.E. Richart, The Strength of Concrete, Its Relation to Cement, Aggregate, and Waters, Bulletin 137, Engineering Experiment Station, University of Illinois, Urbana, 1923, 118 pp.

(2) A. Casagrande, "Classification and Identification of Soils," Transactions, ASCE, 1948, pp. 901-922.

(3) S.B. Hudson and H.F. Waller, Evaluation of Construction Control Procedures: Aggregate Gradation Variations and Effects, National Cooperative Highway Research Program, Report No. 69, 1969, 49 pp.

U.S. Sieve Size	Standard Sieve Opening, mm	Weight Retained, g	Percent Retained	Accumulated Weight Retained, g	Material Finer, %
3 inch	75.00	0	0	0	100.0
2½ inch	63.00	200	2	200	98.0
2 inch	50.00	1,500	15	1,700	83.0
1½ inch	38.10	1,100	11	2,800	72.0
1 inch	25.00	1,700	17	4,500	55.0
3/4 inch	19.00	1,100	11	5,600	44.0
3/8 inch	9.50	1,900	29	8,500	15.0
No. 4	4.75	1,200	12	9,700	3.0
No. 8	2.36	50	0.5	9,750	2.5
No. 16	1.18	50	0.5	9,800	2.0
No. 30	0.60	50	0.5	9,850	1.5
No. 50	0.30	50	0.5	9,900	1.0
No. 100	0.15	30	0.3	9,930	0.7
No. 200	0.075	20	0.2	9,950	0.5
Fines	<.075	50	0.5	10,000	0

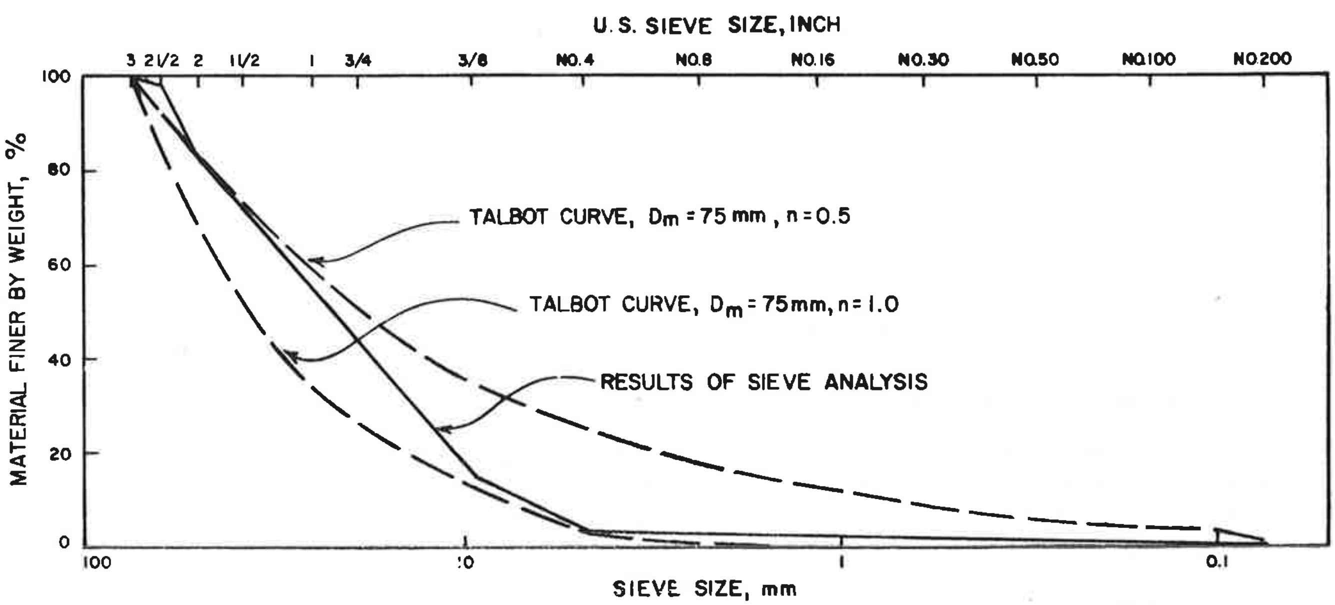


FIGURE 3-1. EXAMPLE OF PARTICLE GRADATION REPRESENTATIONS

FINENESS MODULUS

Sum of individual percentages retained, No. 100 to 3-inch sieve, divided by 100; ratio of adjacent sieve sizes 2.

0.3	No. 100
0.5	No. 50
0.5	No. 30
0.5	No. 16
0.5	No. 8
12.0	No. 4
29.0	3/8 inch
11.0	3/4 inch
11.0	1½ inch
0	3 inch
65.3	

F.M. = 65.3/100 = 0.653

CASAGRANDE PARAMETERS

D_{60} = 28.5 mm

D_{30} = 13.5 mm

D_{10} = 7.0 mm

$$C_u = \frac{28.5}{7.0} = 4.1$$

$$C_c = \frac{(13.5)^2}{(7.0 \times 28.5)} = 0.9$$

GRADATION MODULUS

Sieve Size	Sieve Opening, mm	Mean Diameter, $\bar{d}$, mm	A_i	Separate Weight Fraction, %	Weighted Gradation Modulus
3"	75.00	68.8	-0.33	2.0	-0.007
2½"	63.00	56.2	-.04	15.0	-0.057
2"	50.00	43.8	0.32	11.0	0.036
1½"	38.10	31.1	0.82	17.0	0.139
1"	25.00	21.9	1.33	11.0	0.215
3/4"	19.00	13.7	2.00	29.0	0.58
3/8"	9.50	6.85	3.00	12.0	0.36
No. 4	4.75	3.42	4.00	0.5	0.020
No. 8	2.36	1.70	5.00	0.5	0.035
No. 16	1.18	0.86	6.00	0.5	0.030
No. 30	0.60	0.43	7.00	0.5	0.035
No. 50	0.30	0.22	8.00	0.3	0.024
No. 100	0.15	0.11	9.00	0.2	+0.018
No. 200	0.075			Sample $\bar{A}$ = 1.43	

FIGURE 3-1. EXAMPLE OF PARTICLE GRADATION REPRESENTATIONS (Continued)

where

d_i = the effective diameter of the aggregate particles, in millimeters, within the size fraction.

The effective mean diameter of the particle sizes in a size fraction contained between two sieves may be calculated as

$$d_i = (d_1 - d_2)/\log_e(d_1/d_2)$$

where

d_1 and d_2 = size openings of the coarser and finer sieves, respectively.

The arbitrary constant -- 54.8 mm -- is used because it's the effective mean diameter between the 75-mm (3 inch) and 38.1-mm (1-1/2 inch) sieves. The gradation modulus for the entire sample is calculated as the weighted average of the gradation modulus for each size fraction ($\bar{A}_i$). Any set of sieves may be used. If a standard set of 10 sieves between a 38.1-mm sieve and a No. 200 (0.075-mm) sieve is used, $\bar{A}$ may be simply calculated by adding the percent passing through each sieve and dividing the sum by 100. The calculation of $\bar{A}$ for the gradation curves is shown in Figure 3-1. The single parameter $\bar{A}$ characterizes the average particle size, with higher values indicating finer material.

Electrical and Maintenance Characteristics

Electrical properties of ballast may be measured by using procedures specified in the AAR Signaling Principles and Practices Manual. However, conductivity is infrequently a factor in specifying ballast performance. The index tests that affect ease of maintenance have been discussed in previous paragraphs.

Engineering Tests

In the past 5 to 10 years, significant efforts have been made in the laboratory to measure mechanical properties of ballast. One-dimensional strain (oedometer), triaxial, and direct shear tests have all been employed. These tests provide direct measures of strength and deformability of ballast. However, they require complex equipment and are too costly at this time to serve as index tests for standard use.

3.3 CLASSIFICATION OF BALLAST MATERIALS

At the present time, ballast materials are classified in the railroad literature into one of six broad categories: traprock, granite, limestone, sandstone, slag, and gravel. Only the broadest mineral characteristics are differentiated in this classification system. Thus, all coarse-grained igneous rocks with such varying mineral constituents as granite and diorite, and metamorphic rocks such as gneiss, are grouped as granites. The distinction between the calcium carbonates (i.e., limestone) and magnesium carbonates (i.e., dolomite) is blurred. Therefore, rocks with significant differences in mineralogy and origin are combined into a single category of ballast. This practice has made past correlations between rock type and ballast performance uncertain at best. The previously described ballast classification system may be used for large-scale grouping of ballast materials. However, if ballast materials must be categorized into a system designating similar performance characteristics, that classification system should acknowledge the rock's mineral constituents and its origin. In addition, the system should indicate indices of expected engineering performance characteristics.

Procedures for Ballast Classification and Candidate Index Tests

Table 3-1 lists more than 40 tests that are indicators of ballast performance characteristics. At this time, only limited data exist that relate index test results to in-service ballast performance. A history of evaluated in-service experience with ballast materials is the single most important element lacking in the attempt to relate ballast properties measured in a laboratory to the anticipated service life of ballast. With the current state of knowledge, there's a risk of overlooking a significant test parameter if one fails to use any of the candidate tests. However, it would be expensive and impractical to perform all these tests, expecially since many claim to indicate similar performance characteristics.

Table 3-2 lists 13 index tests that can be used to describe essential material properties of ballast. These tests were selected based on an evaluation of all research and preformance data reviewed for this study. Figure 3-2 is a suggested format for identifying ballast materials samples, including the results of the index tests. Appendix B provides details of the test procedures that are particularly suited to ballast materials.

Each of these tests was chosen for use in describing ballast performance characteristics because:

a. There are some empirical data supporting the correlation between the index test and at least one ballast performance characteristic (preferably more than one characteristic)

b. There is some logical connection between the measured index property and the performance characteristic

TABLE 3-2. RECOMMENDED INDEX TESTS FOR BALLAST CLASSIFICATION

1. Petrographic Analysis [C295]*

   a. Hand sample identification of mineral constituents and percentages, geologic rock classification name (e.g., granite, rhyolite, basalt, granodiorite, gneiss, limestone, dolomite), blast furnace slag, and common rock name (e.g., granite, traprock, limestone).
   b. Abrasion test fines sample: description of minerals present, description of fines as abraded dust or fractured, angular particles.
   c. Polished section or thin section examination if required by petrographer.
   d. Subjective evaluation concerning toughness, hardness, secondary alteration, weatherability of fresh minerals, weatherability of fines, variability of source rock properties.

2. Bulk Specific Gravity [C127]

3. Water Absorption [C127] (Degree of Saturation)

4. Grain Specific Gravity (Total Porosity) [International Society for Rock Mechanics, 1972]

5. Los Angeles Abrasion [C535]: Dry (grading to be specified)
   Wet (add 50% by weight water)

6. Point Load Compressive Strength [International Society for Rock Mechanics, 1974]

7. Mill Abrasion Test [Raymond, 1979]

8. Sulfate Soundness [magnesium sulfate, 5 cycles, 10 cycles (C88)]

9. Reference Density Test [Selig et al., 1977]

10. Flakiness, Elongation Indices [British Standards Institute, Vol. 812]

11. Sieve Analysis [Gradation Modulus (A) and Coefficient of Uniformity (C)] [C136, Hudson and Waller, 1969]

12. Crushing Value [British Standards Institute, Vol. 812]

13. Cementing Test [ConRail-modified]

*ASTM test designation or reference provided in the Bibliography

Sample Number:____________ Date Sampled____________ Date Tested____________

Sample Source:____

Track Location Used:____________________________________

Common Ballast Type:____________________________________

Geologic Rock Classification________________________________

MINERAL CONSTITUENTS		Mineral Type	Percent
Hand Specimen	a.	____________________	______
	b.	____________________	______
	c.	____________________	______
	d.	____________________	______
Fines Examination	a.	____________________	______
	b.	____________________	______
	c.	____________________	______
	d.	____________________	______

Fines Description	Rock Flour	__________________%
	Angular Fragments	__________________%

Petrographic Evaluation ____________________________________

____

____

____

INDEX TEST RESULTS

Bulk Specific Gravity:	_______Mg/m	$MgSO_4$ Soundness - 5 cy:	_____%
		10 cy:	_____%
Water Absorption:	_______%	Reference Density - Uncompacted:	_____Mg/m
Total Porosity:	_______%	Ultimate:	_____Mg/m
Saturation	_______%	Porosity - Uncompacted:	_____%
Los Angeles Abrasion Loss (ASTM C535)		Ultimate:	_____%
		Crushing Value:	_____%
Wet:	_______%		
Grading 2 3 A Dry:	_______%	Flakiness Index:	_____%
Mill Abrasion Loss:	_______%	Elongation Index:	_____%
Point Load Index:	_______MPa	Cementing Value:	_____MPa
PLI Standard Deviation:	_______MPa	Plastic Limit - -No. 100 Fines:	_____%

FIGURE 3-2. BALLAST SAMPLE IDENTIFICATION FORMAT

c. The test is relatively simple and inexpensive to perform; limited special equipment is required

d. The test doesn't require highly specialized operator training, and test results are easy to repeat.

The following discussion outlines the attributes of each of the selected tests.

Petrographic Analysis

Petrographic analysis provides an accurate classification of rock type, source, genesis, and mineralogy. If data related to the in-service performance of various types of ballast materials are to be gathered, it's important to accurately determine mineral and rock types. In addition, the analysis of petrographic data can provide clues to such performance characteristics as hardness, susceptibility to weathering, permeability of disintegrated ballast fines, and porosity. However, petrography is a subjective science, so that the results are not precise; there may be differences in reports by different petrographers. The petrographer must be familiar with the engineering requirements and uses of his analytical results. In addition, petrographic analysis can only be done by experienced personnel, and it is relatively costly.

Bulk Specific Gravity

The bulk specific gravity test (i.e., saturated density) provides an index of total void space within the particles and an indirect measure of toughness. It is a required parameter to calculate inter-particle void fraction from reference density measurements. The test is easy to perform.

Water Absorption

The water absorption test is used to determine saturated porosity. Alone or as saturation, absorption may indicate freeze-thaw resistance. Because determining water absorption aids in evaluating the saturation potential of ballast in the field, the ballast sample should be handled the same way as the in-service ballast. Thus, rather than oven drying, air drying prior to submergence in water is recommended.

Grain Specific Gravity

Grain specific gravity of the common rock-forming minerals varies from approximately 2.5 to 2.8. This possible 10-percent variation leads to a possible 5-percent variation in total void volume and saturation. Because toughness and freeze-thaw resistance may be sensitive to small changes in total porosity and saturation, it's desirable to accurately determine grain specific gravity. In addition, grain specific gravity is a gross indicator of mineral composition.

Los Angeles Abrasion (Dry and Wet)

The dry Los Angeles abrasion test is a popular test for evaluating durability of highway materials and railroad ballast. It's preferred over the Deval procedure because the results are more consistently repeated. Because the test is specified by AREA, there are considerable LAA data on ballasts currently in use. For finer sized ballast, gradation A of ASTM Test C131 may be more representative of the in-track particle gradation. Otherwise, the procedure of ASTM Test C535 is recommended for ballast tests.

The proposal for an abrasion test with water is derived from the experience of British Rail. The wet attrition test (i.e., the Deval test with water) is a principal acceptance test for British Rail ballast. Conducting a wet LAA test will require only minor modifications of the drum door for water tightness. By performing this test with water, it may be possible to develop a correlation with weathering potential, as suggested in 1970 by T.R. West et al. in Tests for Evaluating Degradation of Base Course Aggregates.

Point Load Compressive Strength

Compressive strength is a useful measure of toughness. For ballast, it's sometimes impossible to obtain cylindrical samples for standard compression tests. In addition, cylindrical compression tests require costly diamond coring and trimming of samples.

The point load compression test, as described by the International Society for Rock Mechanics in 1974, permits measurement of a compression strength index on irregular and regular shaped particles. A portable apparatus can be used to perform the test or standard laboratory compression equipment may be used with only minor modification. Since there is no abrasive action involved in this test, the results might be analyzed with the LAA results to try to separate the abrasive and fracture breakdown in the LAA test.

Mill Abrasion

The mill abrasion test provides an index of aggregate abrasion resistance. A sample of 19-mm (3/4 inch) to 38.1-mm (1-1/2 inch) size particles are first rotated with water in a porcelain mill pot. The abrasion loss is then determined by measuring the percentage of fines produced in the pot that pass through a No. 200 U.S. sieve. This test is simple to set up, and has been proposed by Raymond et al. in 1979 as a useful index of the potential for ballast to produce low-permeability fines that can severely limit ballast permeability. No other strictly abrasive test is as suitable or convenient for aggregate samples.

Sulfate Soundness

The sulfate soundness test is proposed as an index of freeze-thaw resistance of ballast. Magnesium sulfate is preferred to sodium sulfate because it's less sensitive to temperature, and test results are more consistently repeated. The test is thought to operate by forming salt crystals within the rock pores which cause internal pressure similar to that experienced due to ice expansion. In addition, samples are subjected to wet-dry and hot-cold cycling.

Current AREA specifications for ballast and the ASTM specification for concrete aggregates (Specification C33) prescribe 5 cycles of sulfate testing. However, some European railroads call for testing of up to 40 cycles. AREA research on ballast has shown that some materials show additional loss beyond 5 cycles, while other materials show negligible loss when subjected to additional exposure. The use of the soundness test for predicting ballast performance has been partially demonstrated(1). The recommendation to evaluate the results at 10 cycles as well as 5 cycles is an attempt to obtain significant additional data at modest additional cost.

Reference Density Test

The reference density test is recommended for three reasons. First, it provides a standard density with which to compare ballast density measured in the field. Second, the test provides data that can be used to predict the range of void ratio to be expected in the field, with compactive efforts ranging from light to heavy. Third, the test may provide data that are related to particle shape and texture properties.

Flakiness, Elongation Indices

Many railroads require limits on the shape of ballast particles. The British flakiness and elongation tests provide data on individual particle shapes without having to intricately measure particle dimensions. If a pair of square and elongated sieves are available, the tests are relatively quick to administer and require little skill.

---

(1) G.P. Raymond and P.N. Gaskin, "Selection and Performance of Railroad Ballast," Railroad Track Mechanics & Technology, Proceedings of a symposium held at Princeton University, April 21-23, 1975.

However, if caliper measurements are substituted for the sieve procedure, special attention must be paid to the definition of sizes and size ratios, as discussed by Lees in 1964(1). The sieve procedure is therefore recommended. Recent research by Dalton (1973), Raymond (1975), and Thompson (1978) have shown limited correlation between the flakiness index and mechanical characteristics of ballast.

Sieve Analysis

Every ballast specification contains restrictions on particle size. The sieve analysis is the standard method of determining grain size for coarse-grained particles. Two parameters are recommended to represent the grain size distribution of ballast:

a. The gradation modulus ($\bar{A}$) -- a measure of the average particle size

b. The coefficient of uniformity ($C_u = D_{60}/D_{10}$) -- a measure of the range of grain sizes in a sample.

Using these two parameters, one can evaluate changes in gradation including (1) whether particle disintegration mostly affects the coarse particles (i.e., $\bar{A}$ increases while C_u decreases), or (2) whether particles of all sizes break down uniformly (i.e., $\bar{A}$ increases while C_u remains unchanged). The graphic grain size distribution curve (see Figure 3-3) also helps in visualizing the physical significance of the sieve analysis test.

Crushing Value

Many European railroads use some type of static or impact crushing test as part of ballast specification. The static test is easily and quickly performed with compression test equipment available in most concrete or materials laboratories. Several studies have shown good correlations between crushing value of ballast and its resistance to disintegration.

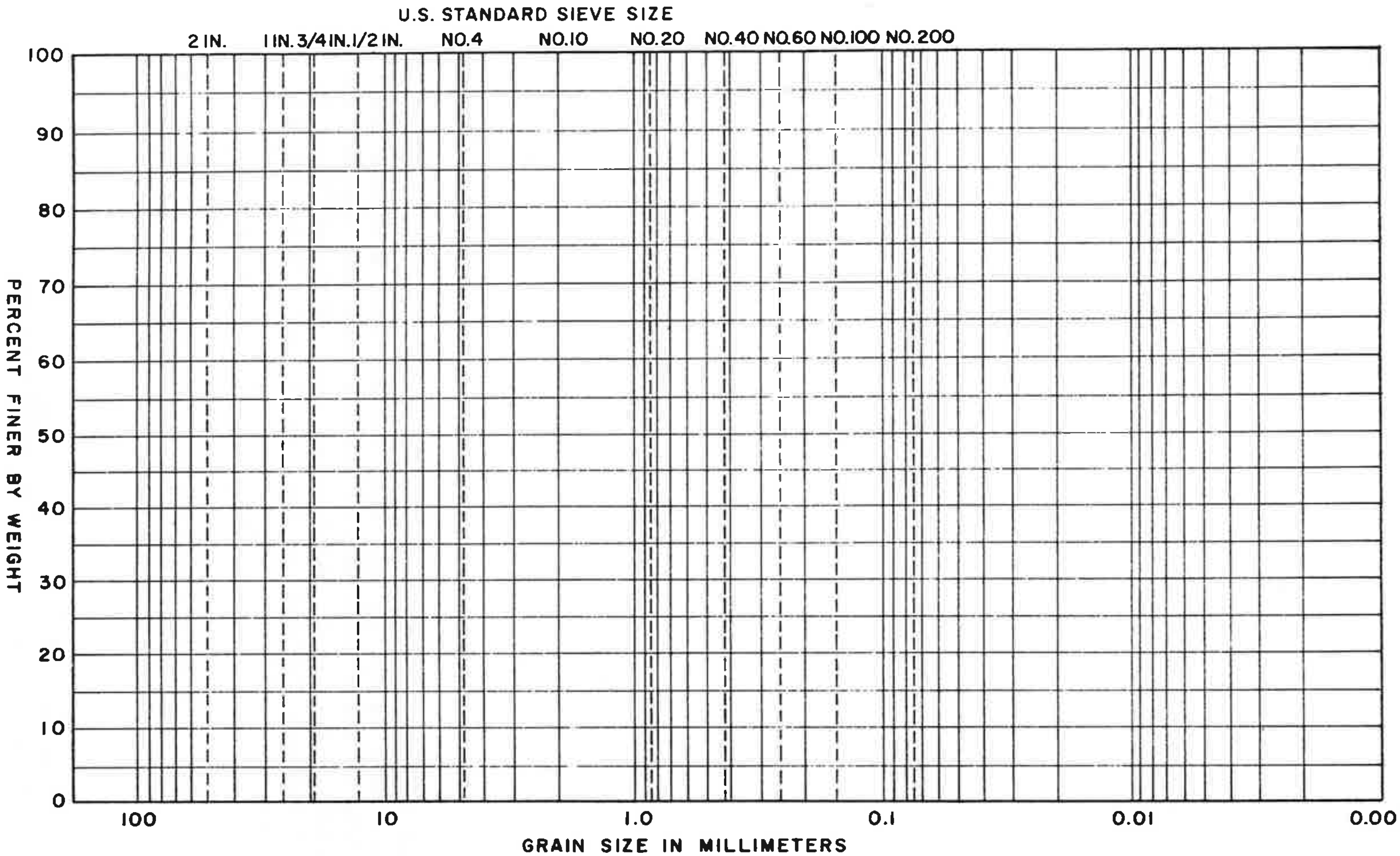
Cementing Value

The cementing value test is used to indicate the tendency of ballast fines to bind into a cohesive cement in the ballast bed. Ballast that exhibits this property loses its ability to drain water and to provide resilient support for the ties leading to excessive tie, rail, and fastener wear. Although there is some question concerning the repeatability of test results(2), it is the only test that offers any promise of indicating the cementing potential of ballast that is both quantitative and can be performed by only moderately trained personnel. Further research into standardizing the cementing value test is required to limit some of the variability in test results observed in previous studies.

---

(1) G. Lees, "The Measurement of Particle Elongation and Flakiness: A Critical Discussion of British Standard and Other Methods," Magazine of Concrete Research, Vol. 16, No. 49, December 1964, pp. 225-230.

(2) F.P. Nichols, Jr., "Review of History of Old Cementing Value Test," National Crushed Stone Association, unpublished, 1975.



COBBLES	GRAVEL		SAND			SILT OR CLAY
	COARSE	FINE	COARSE	MEDIUM	FINE	

UNIFIED SOIL CLASSIFICATION SYSTEM

COEFFICIENT OF UNIFORMITY = D_{60}/D_{10} = ________

GRADATION MODULUS = $\bar{A}$ = ________

FIGURE 3-3. BALLAST SAMPLE GRADATION FORM

Classification of Ballast in Track

Testing - The tests recommended in Table 3-2 should be used to classify and characterize the properties of ballast sampled from track after in-service exposure. Additional tests may also be desirable depending on the purpose of the in-service ballast testing program. The reasons for evaluating in-service ballast must be established prior to the examination. Usually, the purpose is to determine the causes of a deteriorated ballast bed, to collect data on ballast that performs acceptably, or to evaluate ballast, subballast, and subgrade conditions for planning maintenance and rehabilitation measures to preserve or increase service levels.

As with fresh ballast sources, it's necessary to accurately determine ballast composition. Frequently, in-service ballast contains both whole and disintegrated ballast particles and such external contaminants as locomotive sand, waste from hopper cars, soil from surrounding areas, subballast, and subgrade. It may be necessary to collect samples of materials from external sources to aid in accurately identifying the source of the ballast contamination. Confirming these sources may require completion of a more complex petrographic exam including x-ray diffraction or electron microscope studies.

Gradation testing may provide the easiest objective measure of changes in ballast due to service. Ballast disintegration and external fouling will increase the amount of fine particles. Hydrometer analysis, using ASTM Standard Method D422, to determine fine grain size distribution may be useful in defining the gradation characteristics of the fine contaminants as well as Atterberg limits and consolidation/permeability tests.

Sampling - In sampling ballast, the objective is to obtain a specimen that represents the ballast material in stockpile, as delivered, or at various locations in the track. Associated with the considerations of ballast sampling, are the factors of particle segregation and the natural variation of the ballast material.

Segregration of highway aggregates due to handling and stockpiling is the topic of the 1964 NCHRP* Report No. 5, "Effects of Different Methods of Stockpiling Aggregates - Interim Report." Unless specific procedures are used that prevent segregation, it is easy to develop a material source that distributes coarse particles to one section of track while reserving the finer particles for a different section of track. Although the desired particle gradation may have been produced at the production plant, the material delivered to the track may not meet the specified particle size distribution. The above-referenced NCHRP report provides suitable procedures to limit this undesirable effect. Segregation of ballast may also occur during transportation in hopper cars and may not be avoided in track if the ballast is unloaded directly from the car onto the road bed.

*National Cooperative Highway Research Program.

Ballast samples may also show heterogeneity due to variability in the production process, i.e., crushing and screening process. Since ballast is typically produced from natural geologic materials or furnace waste products, the source rock will also exhibit a degree of heterogeneity. All these factors will produce a ballast that differs from place to place and sample to sample even without effects of segregation, in-service loading, and aging.

To obtain samples from ballast stockpiles that are representative of the overall source material, a procedure was developed for use at the FAST track in Pueblo, Colorado, that increases the potential for getting a ballast sample containing a representative collection of particles. The principal is to obtain a sample from several portions of the stockpile where segregation is limited. This requires digging well into the pile rather than taking material that is near the surface, particularly near the top or bottom of a stockpile slope. The following procedure for sampling from stockpiles is recommended:

a. Sample stockpile from four representative faces distributed around the perimeter; sample from two representative heights on each face approximately 1/3 and 2/3 of the height of the face; obtain lower sample first on each face

b. Insert front end loader bucket as deep into stockpile as possible on the face, rotate bucket upright, and withdraw from stockpile

c. Lower bucket to ground level; take one hand shovel full of ballast from at least four equally spaced intervals along the face of the loader bucket.

d. Total sample weight should be at least 100 pounds (50 kg); a sample from each bucket load sould be saved separately so that eight samples will be obtained from each pile

e. Label each sample with at least the following information: source description, date of sample, location of stockpile, compass direction of sampling face, vertical height of sample above ground surface, total height of stockpile

f. If four faces of stockpile are not accessible, sampling locations should be suitably distributed in order to obtain a representative specimen of the ballast stockpile.

It is probably not possible to obtain a representative sample of ballast directly from a hopper car. Segregation will likely leave the coarser ballast fraction near the top of the car while the finer fraction falls near the bottom doors. If ballast is unloaded directly from cars to the track, sampling should obtain ballast that has exited the car immediately after opening of the doors, as the ballast is half unloaded, and ballast that has exited the car at the end of the carload. If it is known what track length a ballast car is unloaded, it may be satisfactory to take a sample of freshly unloaded ballast from the track at a spacing equal to 1/3 to 1/5 of the distance required to completely unload the car. Samples obtained in this manner should weigh a minimum of 200 pounds (100 kg).

In obtaining samples from in-service track it is important to recognize that ballast properties may vary vertically (distance above or below the base of the tie), longitudinally (whether beneath a tie or between two ties), and laterally (whether beneath the rail seat, the shoulder, or at the center of the rails). A procedure was developed for in-service sampling of ballast at FAST which involves obtaining 21 different samples of ballast as defined in Figure 3-4. On samples from tangent track, it may be acceptable to combine some of the samples, such as 1 and 3, 4 and 5, and 10 and 12. To retain the ballast particles at the surface edge of the sample holes, it is recommended that some type of fast setting material be spread in a circle. Plaster of Paris or polyurethane foam are suggested. Each of the sample holes should be approximately 1 foot (0.3 meter) in diameter to obtain a sample volume of approximately 0.5 to 1 cubic foot or 50 to 100 pounds of ballast. It is essential that all fines in the sample hole be collected along with the coarse particles. Each separate sample location should be separately bagged and identified including the date, tie location, and sample location (under tie or within crib; under rail, at center, or in shoulder; and depth above or below the base of the tie). After samples are obtained from the shallow level, excavation to the top of next deeper level should be made carefully to avoid contamination of the deeper ballast material. Be sure to note the total depth of ballast, depth of ballast in the crib, the presence and nature of any subballast or any standing water in the ballast bed. It would also be good practice to obtain samples of the subballast and subgrade soil at representative locations using procedures similar to that for the ballast.

If the ballast layer is thin, less than say 8 to 12 inches, it may be reasonable to take only one set of samples below the base of the ties. However, if the two layer sampling is omitted, it is not possible to determine the vertical distribution of fines within the ballast bed.

Ballast Properties Yielding Acceptable Performance

Presently, what is considered to be acceptable ballast in North America is selected based on specification limits of index test properties. The most widely used specifications, shown in Table 3-3, were published in the AREA Manual. Several American railroads have adopted the AREA specifications, some with modifications to these minimum requirements. European railroads use similar specifications for minimum ballast requirements.

In the past, specification limits were frequently based on the qualities of ballast sources located along the railroad lines; i.e., the limits were set to accept readily available materials. Today's AREA limits apply to railroads located throughout a vast geographic area. These limits are generally considered lenient, thereby including many materials that previously may have been observed to perform poorly in track. In some cases, railroads may purchase materials that don't meet the AREA limits if these materials are available within an acceptable haul distance.

With experience, some railroads have determined which of the ballast sources they have used result in a ballast bed that performs well and which sources of ballast lead to a track that has extraordinary maintenance requirements to sustain service levels. Based on this experience, these railroads can select ballast from sources with which they have had experience and which provide acceptable performance. However, neither the AREA specifications nor past experience by the railroads with specific sources provides direction for selecting desirable ballast materials from sources that have never been used in railroad track.

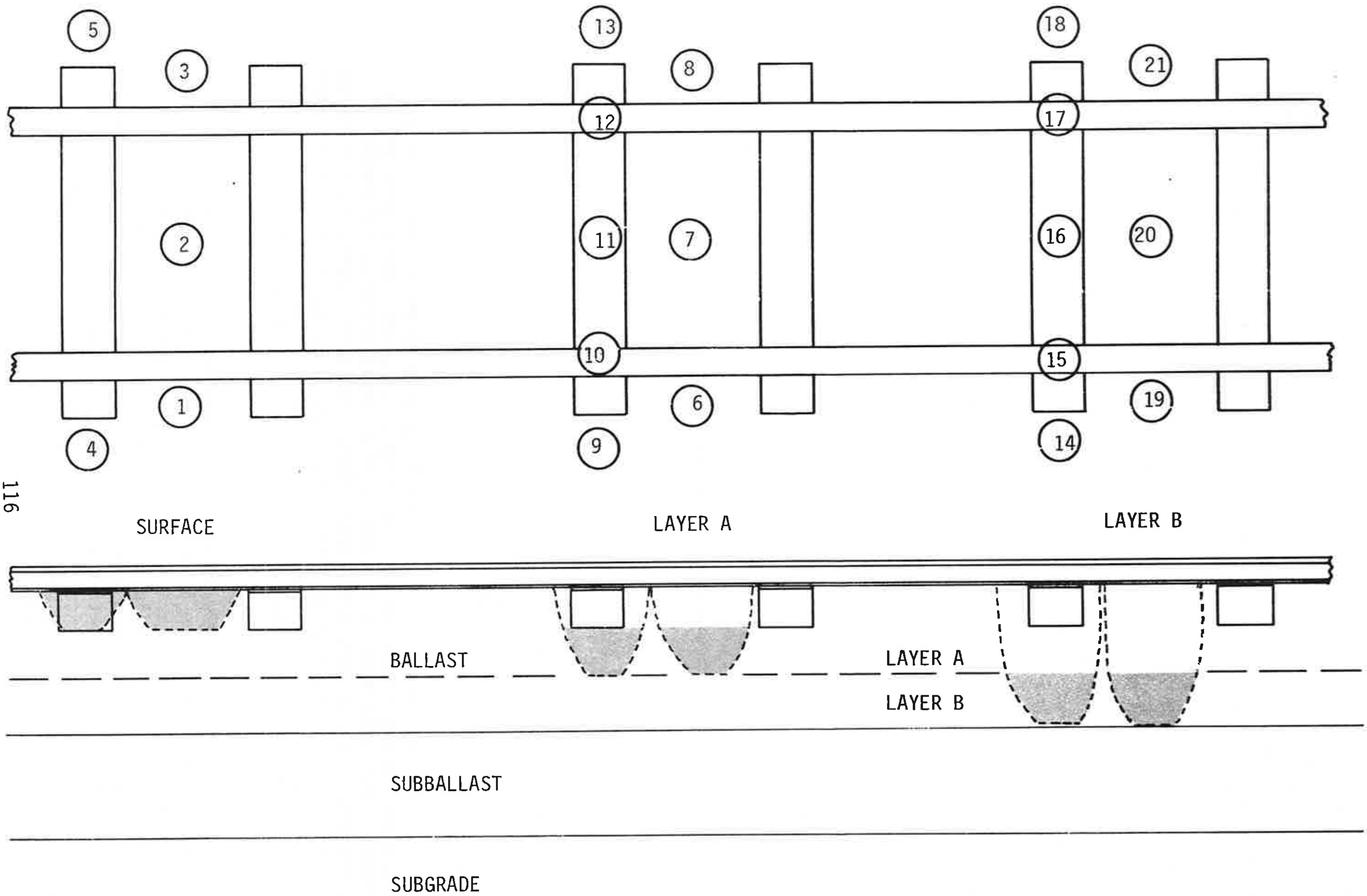


FIGURE 3-4. LOCATIONS FOR SAMPLING IN-SERVICE BALLAST

TABLE 3-3. AREA SPECIFICATIONS FOR PROCESSED STONE AND SLAG BALLAST

Material Characteristics	Limit
Soft and friable pieces	not more than 5.0%
Material finer than a No. 200 sieve	not more than 1.0%
Clay lumps	not more than 0.5%
Los Angeles Abrasion loss	not more than 40.0%
Use standard test grading most nearly representative of the size of ballast specified	
Sodium sulphate soundness loss, 5 cycles	not more than 7.0%
Compacted unit weight - blast furnace slag	not less than 70 pcf
- steel furnace slag	not less than 100 pcf
Flat or elongated particles	not to exceed 5.0%
Grading specified for various standard sizes	
Generally no more than 5 percent to 15 percent by weight passing through a 10-mm (3/8") sieve are permitted. The maximum size generally is 50 mm (2") to 62.5 mm (2 1/2")	

NOTE: No distinction for class or loading rate of track.

From the Manual for Railway Engineering, Chapter 1, Part 2, by the American Railroad Engineering Association. Year of publication: 1978.

The only attempt to establish limits on index properties, which were based on objective evaluations of ballast performance observed at a Canadian National Railroad test section, were made by Raymond in 1975 and 1979. A summary of these recommended index property limits is shown in Table 3-4. In light of the improved mechanical and permeability performance of widely graded ballast, Raymond has recommended a relatively widely graded ballast for mainline track applications as depicted in Figure 3-5. We believe that this attempt by Raymond to use objective ballast performance measures as a guide to ballast selection is the most promising and potentially most reliable means of developing ballast selection guidelines. Although Raymond's suggested limits are admirable, they are restricted by the data base from which they were developed. Further development along this line is urgently needed.

3.4 BALLAST SERVICE LIFE PREDICTIONS

At the present time there are few data, either raw or evaluated, on which to base predictions of ballast service life. A preliminary methodology for prediction of service life of ballast has been developed by Selig et al.(1). However, this methodology only deals with mechanical characteristics of ballast service - progressive settlement due to the application of repeated loads by trains. These predictions are made using relatively high cost, specialized cyclic triaxial test results combined with computerized analytical models. Implementation of such a sophisticated procedure as a practical design tool is impractical, in our opinion. The additional factors of permeability and environmental performance are simply not addressed by this methodology.

To develop a practical method of ballast service life prediction, it is recommended that future research should try to correlate index test results and observations of in-service ballast performance similar to the work reported in 1975 by G.P. Raymond et al. in "Selection and Performance of Railroad Ballast." For these future studies, the suite of index tests recommended in Section 3.3 should be used. The most challenging part of such a study would be to develop procedures to observe and quantify in-service ballast performance. The recommended index tests will serve as a basis for these observations. However, additional measurements of in-service performance such as ballast compression under ties, lateral tie resistance, in-situ ballast seismic velocity and damping, track geometry change, maintenance frequency, and other factors may help provide the input data necessary to develop correlations with index test parameters so that practical track service life predictions can be made in the future. Such studies should consider track operating factors including maximum axle loads, tonnage, temperature, rainfall, subballast and subgrade characteristics and groundwater profile. Further discussions of recommended ballast research are contained in the study final report.

---

(1) E.T. Selig et al., A Theory for Track Maintenance Life Prediction, First Year Final Report prepared for U.S.D.O.T., Office of University Research, Contract DOT-RSPA-DPB-50-79-22, 1979, 183 pp.

TABLE 3-4. AGGREGATE BALLAST SPECIFICATIONS

(For Class A track; maximum speed 100 mph; as FRA Class 6, unlimited tonnage per year)

Los Angeles Abrasion loss (LAA)	not more than 30 to 20%*
Mill abrasion loss (MA)	not more than 4 to 2.5%*
Cementing value	not more than 1.0 to 1.3 MPa (145 to 190 psi)*
Magnesium sulphate soundness loss	not more than 3%
Material finer than No. 200 sieve	not more than 1%
Soft particles	not more than 3%
Clay lumps	not more than 0.5%
Compacted density	not less than 1.76 Mg/m^3 (110 pcf)
Shape factor (index of longest to shortest dimension)	not more than 2

Grading: Moderately broad grading from 60 or 70 mm to 10 m (2½ or 3 inches to 3/8 inch)

*Higher LAA permitted with lower MA. Higher cementing value permitted with lower MA loss as shown below.

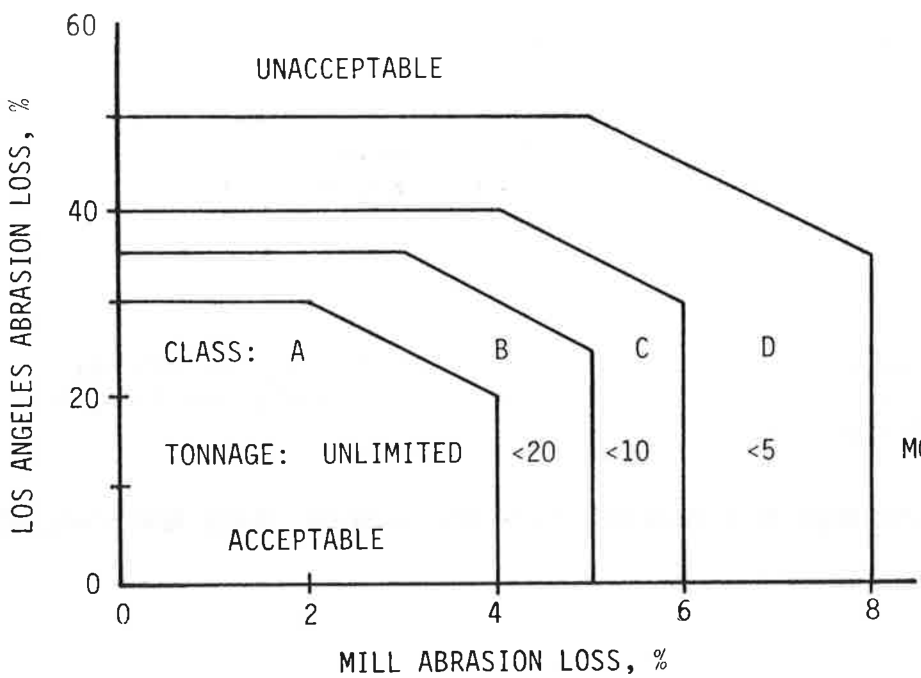
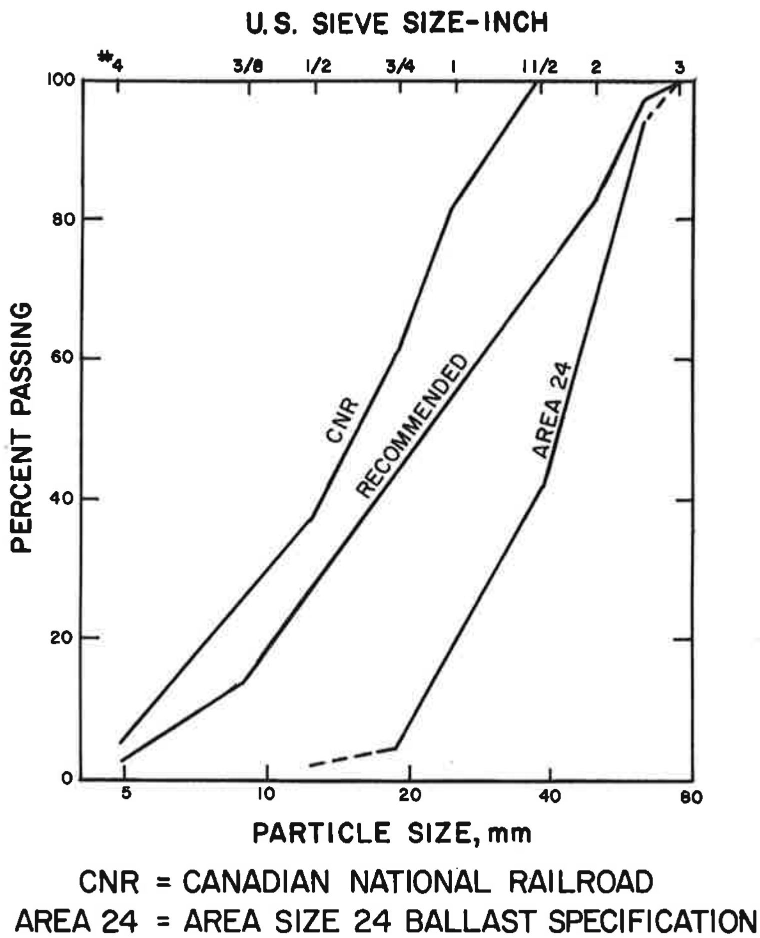


Figure after "Ballast Properties that Affect Ballast Performance," p. 449, by G. P. Raymond, by permission of the American Railway Engineering Association. Year of first publication: 1979.



After "Ballast Properties That Affect Ballast Performance," p. 445, by G.P. Raymond, by permission of the American Railway Engineering Association. Year of first publication: 1979.

FIGURE 3-5. RECOMMENDED MEAN GRADING CURVE FOR MAINLINE TRACK APPLICATIONS

4. SUBBALLAST MATERIALS FOR CONVENTIONAL RAILROAD TRACK

Subballast is the layer of material that is placed between the subgrade and ballast to perform the following functions:

a. Maintain the line and surface of the track

b. Distribute traffic loads from the ballast to limit stress concentrations on the subgrade to an acceptable level

c. Dampen or absorb vibrations generated by the rolling stock on the track structure

d. Prevent mixing of the subgrade and ballast layers

e. Intercept water draining from the ballast and direct it away from the subgrade to ditches or other facilities at the sides of the track

f. Reduce frost penetration into the subgrade.

Some of these functions are similar to top ballast functions. Those functions that are not the same require material properties for subballast that are vastly different from ballast.

Research related to subballast used for railroads has been performed mainly in Europe, Japan and Canada. Extensive research in the related area of highway subbase materials has been carried out in the United States. These research studies provided information relevant to railroad subballast for the recommended practices presented herein.

In the past, conventional railroad track was constructed by placing ballast directly on soil subgrades. The ballast used was crushed rock, natural or crushed gravel, sand and gravel, cinders, or other inexpensive granular material; however, some of these materials distintegrated over time and mixed with the subgrade. New ballast had to be continually placed over the top to maintain track surface and ballast permeability. In old rail lines, the disintegrated ballast forms a layer beneath the new ballast which acts as subballast.

Present track construction practices in North America and abroad use an engineered subballast below the top ballast and above the subgrade because:

a. A properly selected subballast will perform <u>all</u> the subballast functions better than a coarse, crushed rock ballast

b. Subballast material requirements are more readily satisfied than ballast requirements. Candidate subballast materials are normally less costly than ballast materials per equal volume.

The choice of subballast materials used for railroads is frequently based only on the availability of economical local sources; there are limited restrictions on quality. In North America, specifications for subballast materials are most often based on the recommendations contained in Chapter 1, Section 2.10 of the AREA Manual. The AREA gradation recommendation approximates Gradation C of the ASTM specification D1241 for highway subbase material. However, the manual doesn't mention the basis of this gradation recommendation. As discussed below, selection of subballast material should be influenced by the properties of the overlying ballast and the underlying subgrade, and other loading and environmental factors acting on the subballast.

Two types of materials are used for subballast. By far, the most common materials are naturally occurring or processed sand and gravels and crushed natural aggregates or slags. These should be considered as a single class of materials because their engineering behavior is generally understood as cohesionless soils.

The other broad class of subballast behaves as cohesive or cemented soils. Clean, sandy materials may be stabilized with cohesive soil to form a stabilized sand-clay subbase material. Cement or lime-stabilized soils taken from local borrows may be used for subballast if natural or processed aggregates are not economically available. Asphalt-stabilized soil is used for subballast in those rare instances when such a measure is justified economically. Most of the discussion of materials in this chapter centers on granular subballast. For a more complete discussion of stabilized soils used in railroad track, see Section 5 of this report on subgrade stabilization.

Section 4.1 of this chapter presents the performance characteristics of subballast that are necessary to satisfy the required functions. Index tests available to quantitatively evaluate material properties are described in Section 4.2. Finally, Section 4.3 contains a recommended practice for classifying, characterizing, and selecting subballast materials.

4.1 SUBBALLAST PERFORMANCE CHARACTERISTICS

Subballast performance characteristics determine how well the subballast performs its intended functions. These performance characteristics can be divided into four groups:

a. Mechanical -- Representing the ability of the subballast to support the ballast and track under various loading conditions

b. Environmental -- Representing the ability of the subballast to resist alteration due to effects of temperature, infusion of water, etc.

c. Permeability -- Representing the resistance of the subballast to passage of water and soil particles through it

d. Construction -- Representing the ease of placing the subballast in the track in suitable condition to provide the desired performance characteristics.

A discussion of each group of performance characteristics and the material properties that affect performance are presented in the following subsections.

Mechanical Characteristics

Mechanical performance characteristics determine how the subballast will perform the following functions:

a. Maintain the line and surface of the track

b. Distribute loads from the ballast to the subgrade

c. Dampen or absorb transient loads applied to the track structure.

The most significant mechanical performance characteristics are strength and stiffness. The factors that have been shown to have the greatest influence on mechanical performance have been previously described in Sections 2 and 3 of this report, and were reported in greater detail by Robnett et al. in 1975(1) and Selig et al. in 1979(2).

Environmental Characteristics

Due to the ballast cover, subballast is partially protected from the natural environment. However, freeze-thaw and wet-dry cycling are major environmental exposures that will affect subballast and are therefore discussed below.

---

(1) Q.L. Robnett et al., Technical Data Bases Report: Ballast and Foundation Materials Research Program, U.S. Dept. of Transportation, Federal Railroad Administration, Washington, D.C., July 1975, FRA/ORD-76/138, 179 pp.
(2) E.T. Selig et al., Technical Review of Ballast Compaction and Related Topics, Vol. 1, March 1982, 287 pp, Report No. FRA-ORD-81-16.1.

Freeze-Thaw Resistance

Frost penetrating the subballast layer may result in (1) heaving of the layer due to formation of ice lenses, or (2) loss of material strength after thawing. The subballast material may be subjected to frost action if the following conditions are present:

a. Subfreezing temperatures penetrate the subballast material

b. A source of water is available especially below the subballast

c. The subballast material is susceptible to frost degradation. Generally, a material will be subject to frost degradation if more than 3 percent of the material by weight is finer tha 0.02 mm.

The first and second conditions frequently occur because:

a. The subballast layer lies directly below the ballast layer, normally 0.3 m to 0.5 m below the ballast surface. Therefore, it's subjected to subfreezing temperatures several times annually in colder regions.

b. Water available from precipitation percolating through the ballast is present in pockets between the subgrade and subballast and is retained or drawn upward by capillarity in the subballast. Any water in this layer is then subject to freezing.

Further discussion of freeze-thaw resistance is included in Section 2.2 under Environmental Characteristics. The most significant risk with respect to frost action and subballast is not heave, per se, but rather loss of shear strength during thaw. Therefore, subballast must maintain adequate strength when saturated.

Slake Durability

Slake durability is defined as material resistance to weakening and breakdown under alternating wet-dry conditions. Usually, slake durability is only a problem for such lower-quality aggregates as shales and low-grade metamorphic rocks. If weakening and breakdown (i.e., degradation) of the subballast occurs, material properties such as density, friction angle, and gradation will be altered, thereby adversely affecting mechanical and permeability performance. Because the overall durability of subballast may be lower than ballast, it may be necessary to consider slake durability of subballast.

Permeability Characteristics

The primary function of the subballast is to act as a drainage and filter layer between the ballast and subgrade. The desired characteristics of the subballast layer are determined based on subgrade conditions. In almost all cases, water should be conducted away from the subgrade to such drainage facilities as lateral ditches or subdrains. Recently, manufactured plastic products have been used to perform some of the subballast functions. Waterproof membranes such as vinyl or polyethylene have been used to keep surface water away from the subgrade, and porous filter fabrics have been adopted to prevent mixing of the subgrade and ballast. Descriptions of these measures are included in Section 5 of this report describing substructure stabilization methods. The design of drainage facilities that collect the water intercepted by the subballast must be considered in association with subballast permeability. Drainage facility design also is discussed in Section 5 of this report.

When the subgrade is a free-draining material (e.g., a clean, open-graded, granular soil with a deep water table), it may be satisfactory to allow passage of water through the subballast into the subgrade. In all cases, it's paramount that subgrade particles be prevented from mixing with the subballast and that the intrusion of ballast particles into the underlying layers be limited. The gradation and particle size of the subballast will determine how much void space is available for the passage of water and fine soil particles. The broadness of the subballast gradation range will determine its susceptibility to ballast intrusion.

Construction Characteristics

There are two performance characteristics that affect operations during the construction of the subballast layer:

a. The ease with which materials can be placed to obtain a satisfactory in-place material physical state (e.g., the degree of compaction)

b. The ability of the material to sustain the loadings and exposure during construction without changing material properties.

The physical state of granular fill is a function of placement water content, mechanical effort used during compaction, and stability of the base on which the fill is being placed. Water content plays an important role in both lubricating soils to enable them to achieve compaction and binding soils together by water surface tension to effectively contain compacted soils and aggregates. Further discussion of compaction is included in Section 5.

The effect of the soil moisture content is a function of the size of the pore spaces within the soil which, in turn, are a function of soil gradation. Therefore, the most significant factor with respect to ease of compaction of soil and aggregate mixtures is the grain size. If a soil or aggregate mixture contains significant quantities of fines (material finer than the No. 200 sieve), the plasticity of the fines will have an important influence on the effects of soil moisture. Plasticity -- as usually measured by the Atterberg Limits test -- indicates the effect of soil moisture on the consistency of the finer soil fraction (i.e., material passing through a No. 40 sieve). Materials with a higher fines content will require more water for compaction and will achieve generally lower maximum density and lower strength. High fines content will also lead to frost susceptibility.

Subballast is located at a depth where stresses due to train loadings are not extreme. Rather, the stresses induced by heavy compaction equipment may be the greatest mechanical loading received by the subballast material. Therefore, the resistance of these materials to degradation during compaction is important if they are to maintain a desired gradation through construction. Degradation or disintegration of subballast is a function of the hardness and toughness of the aggregate particles. (These properties were previously discussed in Section 3).

Subballast gradation may also change because of handling procedures used prior to final placement which can lead to segregation and degradation. Subballast is normally borrowed from one location, processed at a central plant, transported to the construction site in either rail or highway vehicles, and placed on the subgrade either directly from trucks or by such equipment as bulldozers and loaders. Segregation and degradation are functions of the transportation process (i.e., the intensity and duration of agitation induced during hauling) and the gradational characteristics of the subballast (e.g., broadly graded materials may be less susceptible to segregation than gap-graded materials). The handling aspects of aggregates have been extensively studied with respect to use in highway construction.

4.2 SUBBALLAST INDEX TESTS

Material properties that influence subballast performance characteristics are classified as:

a. Mechanical
   - Reference density and in-situ density
   - Gradation/particle size
   - Plasticity of fines
   - Confining pressure
   - Saturation/water content

b. Environmental
   - Frost-heave potential
   - Slake durability

c. Permeability
  Gradation/particle size
  Saturation/water content

d. Construction
  Abrasion resistance
  Compaction-water content characteristics.

Index tests for subgrade soils and ballast are discussed in both Sections 2 and 3. Because many of the tests used to indicate subballast material properties are the same as those previously recommended for soils and ballast, refer to these chapters for detailed discussions of these tests. The following discussion covers only those tests or aspects of the tests that are peculiar to subballast.

Reference Unit Weight/Compaction-Water Content Characteristics

In the 1982 report, Field Methods for Ballast Physical State Measurements, Selig et al. reviewed tests for use in determining reference density of earth materials. Generally, the vibratory, kneading and impact compaction methods may be employed, with impact methods being the most commonly used. Standard methods for the Proctor impact compaction test are set forth by ASTM as standard tests D698 and D1557.

The Proctor test requires compaction of soil into a standard sized cylindrical mold using a specified compaction energy (hammer weight, height of fall, number of blows). Because the soil moisture content influences compacted density, the procedure is repeated for different water contents until a maximum dry density is achieved. Results are frequently plotted showing dry unit weight versus water content (see Figure 4-1) and the maximum dry weight and optimum water content at which maximum unit weight is achieved are reported. Normally molds of 102 mm (4 inches) or 152 mm (6 inches) in diameter are used with hammers of 2.2kg (5 pounds) or 4.5kg (10 pounds) weight.

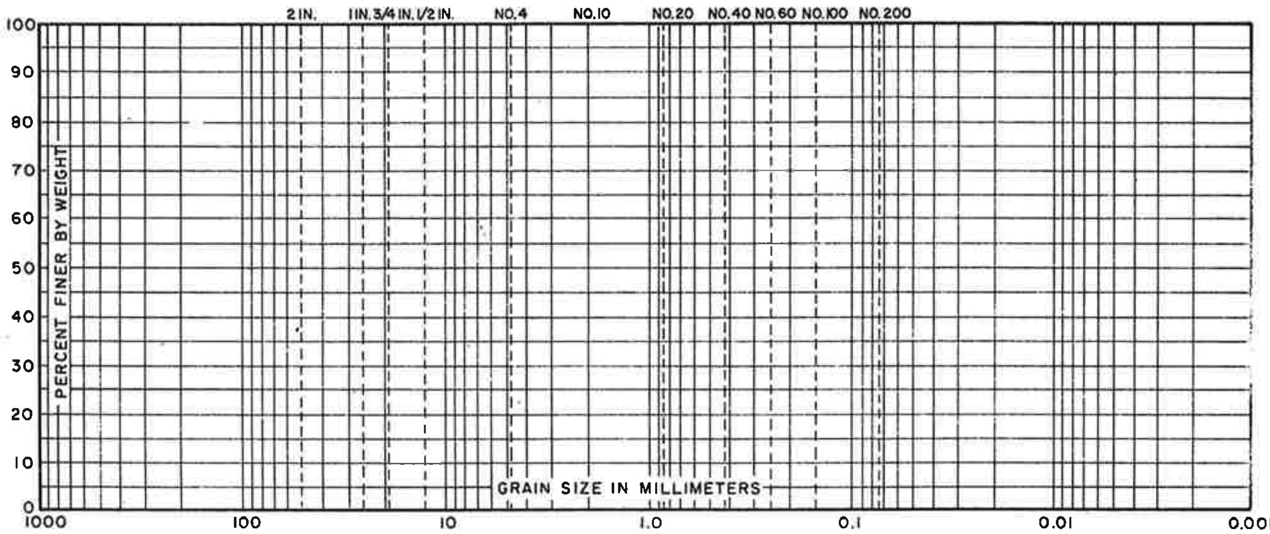
The Proctor compaction test will provide the following useful information:

a. The optimum water content at which the material can be readily compacted to a high density

b. A compaction curve (see Figure 4-1), which indicates the sensitivity of the placement process to initial water content, and provides a guide for limits of placement water content permitted in the field

c. A reference unit weight with which to compare in-situ densities measured in track.

a) GRAIN SIZE DISTRIBUTION

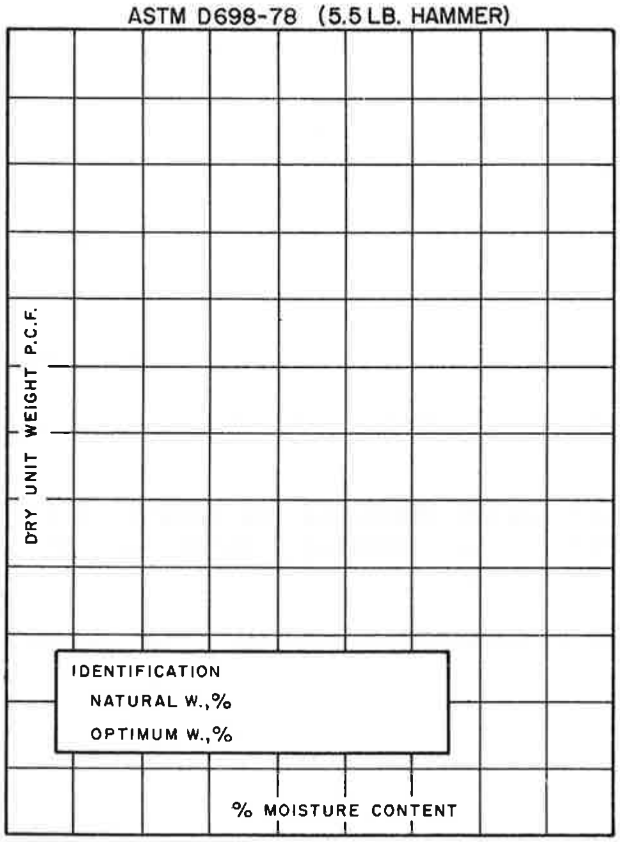


COBBLES	GRAVEL		SAND			SILT OR CLAY
	COARSE	FINE	COARSE	MEDIUM	FINE	

UNIFIED SOIL CLASSIFICATION SYSTEM

b) COMPACTION

ASTM D698-78 (5.5 LB. HAMMER)



c) INDEX PARAMETERS

SAMPLE ______________________

SOURCE ______________________

____

CLASSIFICATION ______________________

____

____

IN-PLACE WATER CONTENT ____________ %

IN-PLACE DENSITY ____________ PCF

PLASTIC LIMIT ____________ %

LIQUID LIMIT ____________ %

PLASTICITY INDEX ____________ %

LOS ANGELES ABRASION

DRY ____________ %

WET ____________ %

REMARKS (STABILIZER) ______________________

____

____

____

SUBGRADE ______________________

____

____

BALLAST ______________________

____

____

FIGURE 4-1. SUBBALLAST CLASSIFICATION FORM

In-Situ Unit Weight

In-situ unit weight is used to indicate the condition of materials in track, and as an index of the confinement. The in-situ dry unit weight is expressed as a percentage of the reference maximum dry unit weight. This is an index of the field compaction quality and of mechanical performance.

In 1977, Selig et al. reviewed two general methods of measuring in-situ unit weight: displacement and nuclear methods. The displacement methods generally use materials of known density -- either sand poured directly into an excavated hole or liquid within a membrane or balloon -- to measure the volume of material removed. The excavated material is then weighed and divided by the in-situ volume to calculate total unit weight, and the water content is subsequently determined to compute dry unit weight.

If the in-situ material contains large voids, sand that is used to refill the hole will escape through the voids to volumes outside the excavated hole. Therefore, the method may yield an excessive volume and a lower than accurate in-situ unit weight. The water displacement method also has limitations with regard to coarse materials with angular particles, because the particles tend to puncture the membrane. If a thick membrane is used, the membrane may not conform to the irregular limits of the excavated hole. Thus, the volume measured will be less than the actual volume, yielding too large a measured unit weight.

The nuclear density procedure uses the transmitted or refracted intensity of gamma rays to determine the density of materials. The nuclear gauges are also limited on materials with coarse, open voids because the measured density may depend on whether the source and the receiver are placed over a solid particle or over a void. The water content of materials may be similarly determined by measuring the intensity of neutron beams. ASTM provides standard methods of executing all of these tests (D1556, D2167, and D2922).

Slake Durability

Slake durability describes the resistance of a material to disintegration under the action of alternate wetting and drying cycles. Slaking is normally important only for low-grade sedimentary rock aggregates (e.g., shale, mudstone, and siltstone). If a material has low slake durability, it may be reduced to plastic soil in a short period of time in track.

No test procedure has been accepted universally for providing a slaking index. In 1972, the International Society for Rock Mechanics tentatively adopted a procedure that involves rotating aggregate samples in a drum of 2 mm (i.e., a U.S. No. 10 sieve) mesh immersed in water for 10 minutes followed by oven drying at 105 degrees C. The following is a slaking index based on the amount of material retained in the mesh cylinder after completing two wet-dry cycles:

SLAKE DURABILITY INDEX

Qualitative Durability	Percent Retained
Very low	0 - 30
Low	30 - 60
Medium	60 - 85
Medium High	85 - 95
High	95 - 98
Very High	98 - 100

There are few field data relating the results of slake durability tests to performance of aggregates in compacted fills. However, it's suggested that an aggregate with a slake durability index of less than 70 percent be carefully evaluated prior to use in an embankment. Because subballast has ready access to water and is subjected to cyclic stresses, it may be wise to restrict subballast to at least the medium high to high range of slake durability.

Other index tests, such as the sulphate soundness and wet and dry Los Angeles abrasion tests, may provide an index of slake durability for the more-durable materials. Both of these tests introduce more severe exposures than the slaking test, and would tend to severely disintegrate materials of medium and low slake durability.

4.3 CLASSIFICATION, CHARACTERIZATION, AND SELECTION OF SUBBALLAST

Subballast materials may be grouped into two broad classes: natural or processed granular materials, and stabilized soils. The source of materials should always be noted; e.g., bank run sand and gravel, crusher run slag, lime stabilized clay. As natural granular materials, it's recommended that the combined Unified and Burmister classification systems be adopted, as described in Section 2.4 of this report. If stabilized subballast is used, the Unified and Burmister soil classifications should be complemented by a description of the type and quantity of the stabilizer.

The following index tests are recommended for characterizing subballast:

a. Particle-Size Analysis (D422)*

b. Moisture-Density Relation (D1557, D698)

c. Liquid and Plastic Limits (if applicable) (D423, D424)

d. Los Angeles Abrasion Resistance (C131)

---

* ASTM designation, <u>Annual Book of ASTM Standards</u>, Part 19 and Part 14, 1979 or latest.

A suggested form for reporting subballast classification and index characteristics is shown in Figure 4-1.

It may be necessary to perform additional index tests -- including the slake durability, sulphate soundness, or frost-heave test -- when local experience with candidate materials indicates the potential for an unsatisfactory reaction to the environment. For stabilized soils, laboratory tests such as a series of unconfined compression tests are ordinarily required to select a suitable soil stabilizer and stabilizer content. The Los Angeles abrasion test may not be needed when experience with local materials has shown that construction durability is not in question.

The subballast functions listed in the introduction to this section relate to three general classes: the mechanical (functions 1,2, and 3), permeability (4 and 5), and environmental (6) performances of the subballast. Based on highway and foundation engineering experience, the mechanical requirements for subballast can normally be satisfied by choosing a well-graded granular material (GW or SW) with relatively broad gradation (C_u>10) that can be placed readily at a suitable physical state (i.e., a high percentage of the maximum Proctor density). It is generally accepted that frost degradation can be avoided by choosing a material with less than 10 percent fines (passing a No. 200 sieve) and with less than 3 percent finer than 0.02 mm.

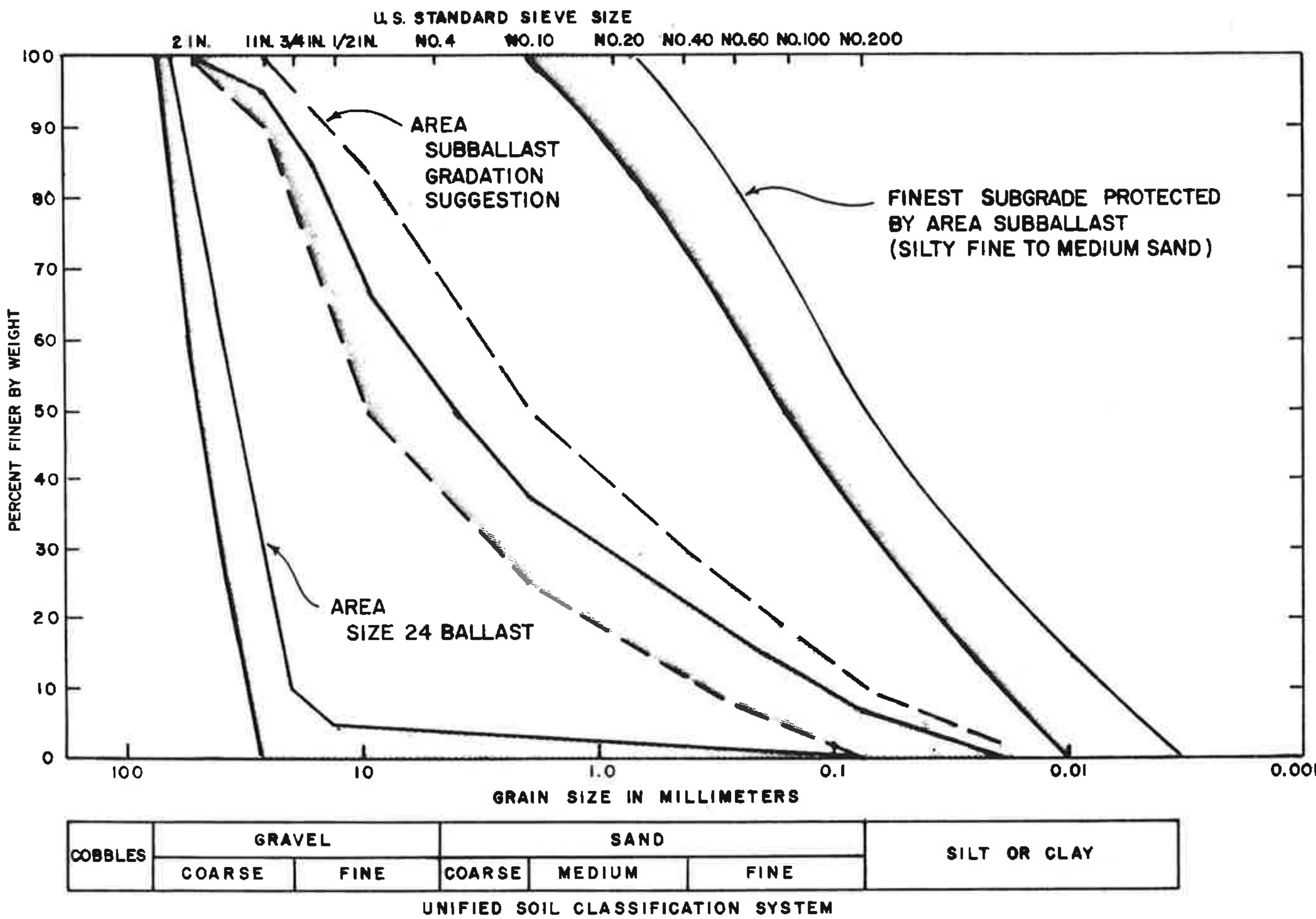
However, the criteria for selecting materials to satisfy the permeability and separation requirements are not well established. The most common recommendation is the AREA suggestion based on ASTM Specification D1241. These gradation requirements are shown in Figure 4-2. As discussed below, this single material gradation specification can be expected to perform the required separation function only for a limited range of subgrade particle sizes.

One approach for selecting subballast gradation has been to adopt the criteria for graded granular filters recommended by Bertram, Terzaghi,and Casagrande in 1940 in An Experimental Investigation of Protective Filters, and suggested in the 1978 AREA manual (Chapter 1, Section 1.2.5.3). These criteria were developed to provide materials that will preclude the erosion of finer soil into the granular filter while providing a layer of significantly higher permeability than the adjacent soil which is drained by the filter.

There are two significant differences between the selection of a graded filter for drainage and the selection of a suitable subballast. First, subballast is not intended to drain the underlying subgrade. Thus, there is no requirement that the permeabillity be greater than the underlying subgrade. In 1961, Schramm(1), based on the practice of the German Federal Railway, stated that a subballast should have a low permeability (10^{-3} to 10^{-4} cm/sec) to keep surface water from reaching the subgrade regardless of subgrade permeability. However, frost-susceptible materials should still be avoided.

---

(1) G. Schramm, Permanent Way Technique and Permanent Way Economy, Otto Elsner Verlagsgesellschaft, Darmstadt, 1961, pp. 97-116, 203-218.



$$\frac{D_{15}\,(\text{SUBBALLAST})}{D_{85}\,(\text{SUBGRADE})} = \frac{0.2\text{mm}}{0.5\text{mm}} = 0.4 \qquad \frac{D_{15}\,(\text{BALLAST})}{D_{85}\,(\text{SUBBALLAST})} = \frac{25}{16} = 1.6$$

$$\frac{D_{50}\,(\text{SUBBALLAST})}{D_{50}\,(\text{SUBGRADE})} = \frac{4\text{mm}}{0.16\text{mm}} = 25 \qquad \frac{D_{50}\,(\text{BALLAST})}{D_{50}\,(\text{SUBBALLAST})} = \frac{40}{4} = 10$$

$$C_u = \frac{D_{60}\,(\text{SUBBALLAST})}{D_{10}\,(\text{SUBBALLAST})} = \frac{6.5\text{mm}}{0.13\text{mm}} = 50$$

FIGURE 4-2. AREA GRADATION CURVES FOR SELECTION OF SUBBALLAST

The second difference between filters and subballast deals with the consequences of a failure. If a graded filter -- e.g., one installed in a dam -- fails to prevent piping or erosion of surrounding finer soil particles, a rapid and even catastrophic failure may result, which could involve loss of the entire dam structure. Alternatively, the filter may become clogged with fines and fail to operate. (This development can have equally serious consequences.) If there is a failure in the performance of a subballast blanket so that mixing with the underlying subgrade occurs, track geometry would deteriorate by a faster rate than desirable. However, the condition would not be considered to be a rapid failure -- in fact, the performance would still be better than if no subballast were used at all. Therefore, it's appropriate to use a lower factor-of-safety in the selection of subballast than in the selection of graded filters.

The criteria for filters originally set forth by Bertram and expanded by the U.S. Army Corps of Engineers and U.S. Bureau of Reclamation are as follows:

(a.)	$\dfrac{D_{15}\ \text{(filter)}}{D_{85}\ \text{(subgrade)}}$	$<$ 4 to 5	(Prevent piping)
(b.)	$\dfrac{D_{50}\ \text{(filter)}}{D_{50}\ \text{(subgrade)}}$	$<$ 25	(Provide uniform grading ratio; avoid gap-graded filters)
(c.)	$C_u\ \text{(filter)} = \dfrac{D_{60}}{D_{10}}$	$<$ 20	(Limit segregation during handling)

where D_n is the sieve size that passes n weight percent of the total material sample.

The piping ratio limit, (a) above, was developed using a factor-of-safety appropriate for dams. Because a lower factor-of-safety may be considered for subballast, a piping ratio limit of 6 to 8 is suggested, with higher values permitted over medium to highly plastic clay subgrades and for well-graded, angular sand and gravel subballasts.

There are few field or laboratory data on which to base criteria for subballast gradation, so that any recommendation must be verified in the field. In his address to the AREA annual meeting in 1979, Raymond proposed a laboratory test to study this problem. The test uses a cyclicly loaded footing bearing on a layer of subballast over a layer of subgrade soil. The experiment uses the thickness of mixed material produced after a fixed number of loading cycles as a measure of the separation capability of the subballast.

Subject to verification, and in consideration of the above discussion, the following criteria are recommended for the selection of granular subballast gradation:

(a) $\frac{D_{15}\ (\text{subballast})}{D_{85}\ (\text{subgrade})}$ <6 to 8

(b) $\frac{D_{50}\ (\text{subballast})}{D_{50}\ (\text{subgrade})}$ <25

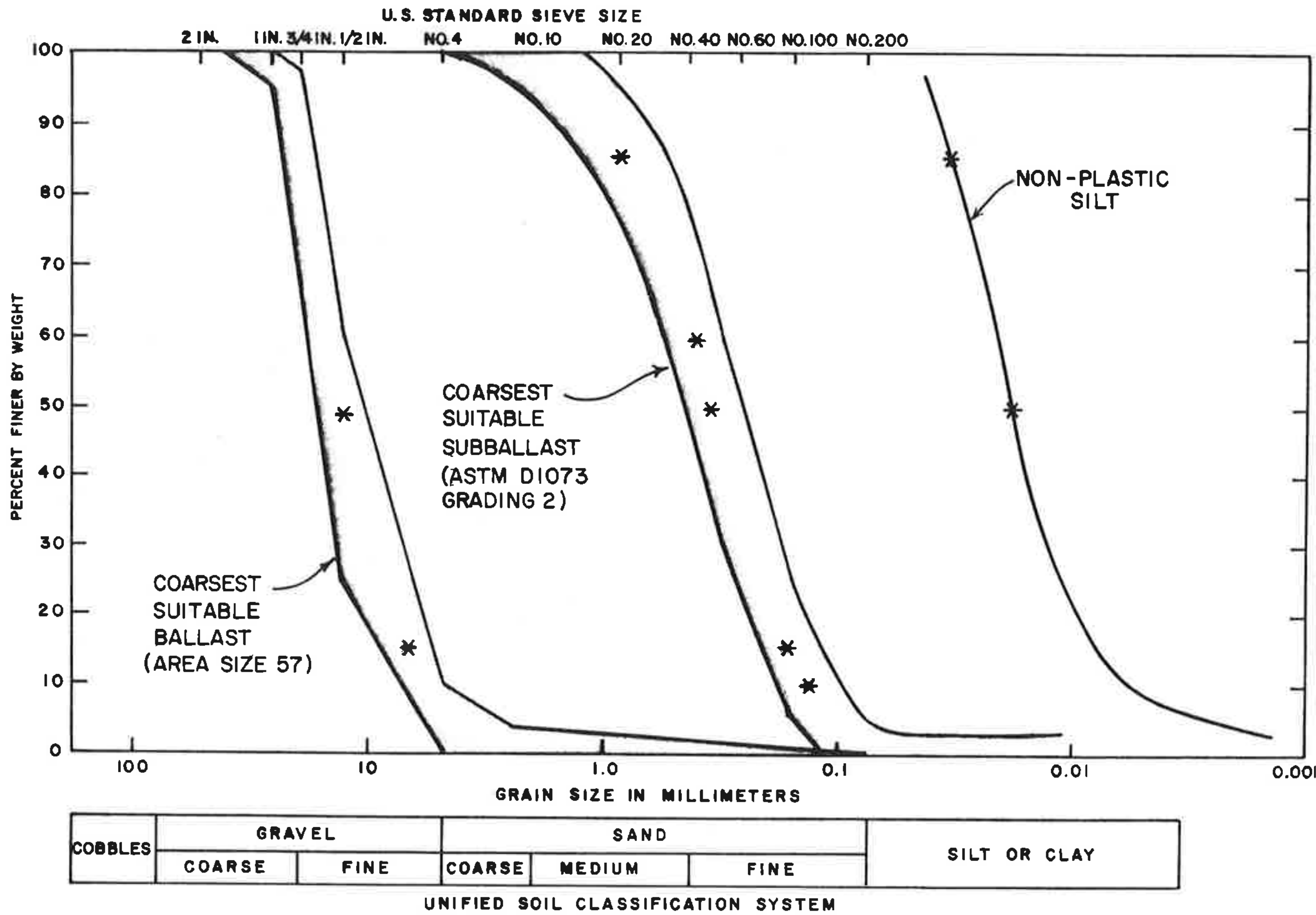
(c) C_u (filter) = $\frac{D_{60}\ (\text{subballast})}{D_{10}\ (\text{subballast})}$ = Between 10 and 20

It's also necessary to keep the ballast from intruding into the subballast. Therefore, it's recommended that criteria (a) and (b) above be satisfied for the ballast-subballast interface, where ballast now replaces subballast and subballast replaces subgrade in the equations.

Using these criteria, one can examine the AREA-suggested subballast gradation and evaluate the subgrade and ballast types for which this material would make an effective subballast. As shown in Figure 4-2, the finest subgrade soil that this subballast can protect is a silty fine to medium sand (SM). The limit on the minimum subgrade particle size is based primarily on criterion (b). With respect to the ballast, the subballast gradation suggested by AREA satisfies the intrusion criterion for even the coarsest of AREA ballast gradation, Size 24, as shown in the figure.

A nonplastic, uniformly graded silt is very susceptible to pumping and requires careful protection with subballast or other filter. The gradation curve for a sample of such material derived from a glacial lake deposit in Rhode Island is shown in Figure 4-3. Also shown are the coarsest suitable subballast and ballast gradations that can prevent mixing of the various layers, based on the above criteria. The actual subballast gradation shown is based on the ASTM specification for bituminous concrete sand, D1073, Grading 2, from Part 15 of the ASTM Standards, 1979. The ballast grading shown is Size 57 ballast, which is finer than the acceptable ballast gradation for most main line applications. Therefore, this two-layer track substructure system would not be suitable over such a silt subgrade for most railroads. To provide a satisfactory blanket over the silt, it's necessary to construct a double layer subballast blanket such as the bituminous concrete sand covered by well-graded sand and gravel blanket under coarse ballast. Such a three-layer system will prevent mixing of the subgrade, subballast, and ballast, and will permit use of a coarse ballast gradation. Alternatively, filter fabric might be placed beneath the subballast to prevent movement of the silt into a subballast of broadly graded sand and gravel. Use of filter fabric is discussed in greater detail in Section 5.

Further experience with the use of subballast on North American railroads is needed to verify criteria for selecting subballast materials.



$$\frac{D_{15}\,(\text{SUBBALLAST})}{D_{85}\,(\text{SUBGRADE})} = \frac{0.15\text{mm}}{0.03\text{mm}} = 5 \qquad \frac{D_{15}\,(\text{BALLAST})}{D_{85}\,(\text{SUBBALLAST})} = \frac{6.5}{0.8} = 8$$

$$\frac{D_{50}\,(\text{SUBBALLAST})}{D_{50}\,(\text{SUBGRADE})} = \frac{0.33\text{mm}}{0.018\text{mm}} = 18 \qquad \frac{D_{50}\,(\text{BALLAST})}{D_{50}\,(\text{SUBBALLAST})} = \frac{12}{0.33} = \underset{(\text{HIGH})}{36}$$

$$C_u = \frac{D_{60}\,(\text{SUBBALLAST})}{D_{10}\,(\text{SUBBALLAST})} = \frac{0.4\text{mm}}{0.13\text{mm}} = 3$$

FIGURE 4-3. EXAMPLE OF GRADATION CURVES FOR SELECTION OF SUBBALLAST

5. SUBSTRUCTURE STABILIZATION METHODS

The functions and performance of railroad track subgrades are discussed in detail in the introduction to Section 2 and in Section 2.2. The prime function of the subgrade is to support the track while maintaining acceptably small displacements. The ability of the subgrade to perform this function is termed subgrade performance. Section 2 provides some suggested practices for characterizing the performance characteristics of subgrade earth materials in order to provide a basis for substructure design and evaluation.

Substructure stabilization methods are used to improve the performance characteristics of substructure elements. The stabilization methods addressed herein are intended to treat the subgrade and foundation elements of the substructure primarily, although some of the methods may be applied to the subballast. Subgrade stabilization may also result in secondary improvement in the performance of the top ballast layer such as by reducing ballast fouling. The types of subgrade performance improvement that can be realized by stabilization procedures include increased shear strength and stiffness, reduced frost heave and swelling action, and reduced pumping of subgrade fines into the ballast layer. Some additional subgrade problems that may be treated by stabilization methods are described in Section 2.7.

Stabilization methods may be implemented at three stages of railroad track development: new construction, track rehabilitation or upgrade, and as part of regular track maintenance. The benefits that can be derived from improving subgrade performance characteristics are reduced requirements for ballast and subballast layer thickness; reduced displacements due to environmental action, such as those caused by frost heave and clay swelling; and generally reduced maintenance requirements. For existing railroad track, the stabilization methods of greatest potential value may be those that require minimal disruption of the track and train operations to achieve stabilization.

Design of substructure stabilization programs requires detailed study of each situation and should be carried out by those experienced in both railroad track and geotechnical engineering. It is not possible to provide sufficient details of each stabilization method within the scope of this report to permit a final evaluation of stabilization method potential for a particular project. Rather, the intent of this section is to provide sufficient information to identify those stabilization methods that are potentially applicable to a particular situation and warrant further study. A classified bibliography related to stabilization is provided in Appendix D to provide sources of additional information.

The substructure stabilization methods described in this section have been selected because of one or more of the following reasons:

a. They have been successfully applied to railroad track.

b. They can be carried out with limited disruption of track and train operations.

c. They have been successfully applied to other types of structures such as pavements or buildings.

d. They are economical, practical, and have potential for track subgrade stabilization.

Section 5.1 discusses drainage methods, probably the most important aspect of track design with respect to maintaining track performance. Section 5.2 describes methods that stabilize soils in-place and therefore require limited disruption of the track in some cases. Section 5.3 describes layer inserts that can be placed in the track substructure either during construction or in association with rehabilitation procedures such as ballast undercutting. Section 5.4 describes compaction and admixture stabilization techniques that are mainly applicable to new and reconstructed track substructure. Section 5.5 describes methods of embankment and slope treatment mainly used to treat deep-seated subgrade and foundation deficiencies. The final section provides a preliminary basis for selecting subgrade stabilization methods to treat specific substructure performance problems.

5.1 DRAINAGE

Providing drainage adjacent to and beneath railroad track is the most important and effective method to maintain and improve subgrade performance and embankment stability. As discussed in Section 2.7, virtually all near-surface subgrade problems are caused by or aggravated by the presence of water near the track surface. Draining embankments or adjacent cut slopes also may alleviate deep-seated stability problems by reducing driving moments, reducing pore pressures, and increasing shear strength of the soil.

Application

Excess water in the track substructure is a principal factor in the development of all near-surface subgrade problems including mud pumping, ballast pockets, squeezes, frost heaving, and swelling of clays. The sources of water in the subgrade include surface water, groundwater, infiltration through the ballast, and capillary action. Embankment surface sloughs and erosion, which may lead to a deterioration of track support, and deep-seated stability problems, including slope or embankment instability and lateral creep, can be alleviated by a properly designed and installed drainage system.

However, drainage installations are more likely to be used to improve the performance of subgrade afflicted by near-surface problems.

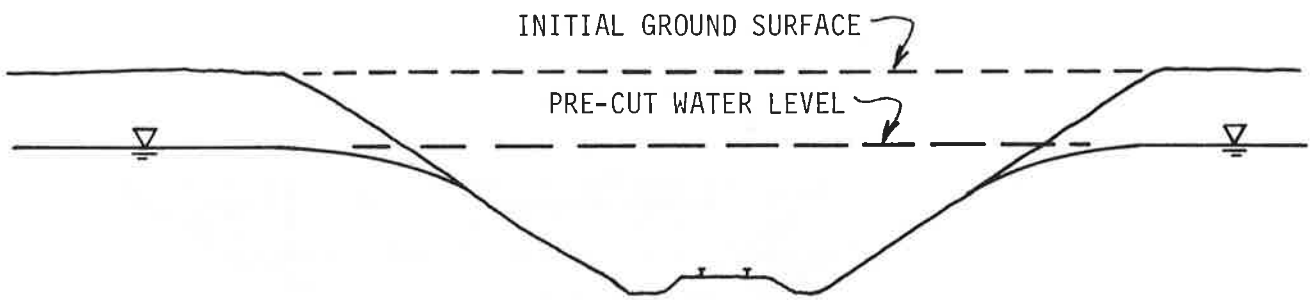
Figures 5-1 through 5-3 illustrate some cases where subgrade drainage is especially important. Figure 5-1 illustrates a cut below the natural groundwater level. Depending upon the specific soil profile, three conditions may arise as a result of the cut -- slope instability, surface sloughs, and a wet subgrade induced by groundwater exiting on the surface. The lower portion of Figure 5-1 shows how several drainage measures might alleviate these problems.

Figure 5-2 depicts a case where drainage may be a concern even when the groundwater level is well below the track surface elevation. In this case, excess water in the subgrade is caused by infiltration through the ballast and subballast. Water from precipitation is trapped in a "bathtub" by the relatively impermeable subgrade soils. The highly permeable ballast allows water to seep in much more rapidly than it can be carried away by drainage through the natural subgrade. The ballast and subballast become saturated with water, as does the top of the subgrade. This "bathtub effect" has been observed to be common in roadway construction. The most effective correction for this condition is to raise the track. Figure 5-3 illustrates a similar case, where surface infiltration can cause a subgrade to become saturated and weakened in an elevated embankment. As can be seen in all these figures, providing or improving drainage below track may not be easy to accomplish.

Erosion is indirectly related to substructure stabilization but will not be treated in great detail in this section. It should be noted that good surface drainage and embankment protection techniques -- such as seeding, sodding, and rip rap -- have been used successfully to limit the damage caused by erosive forces.

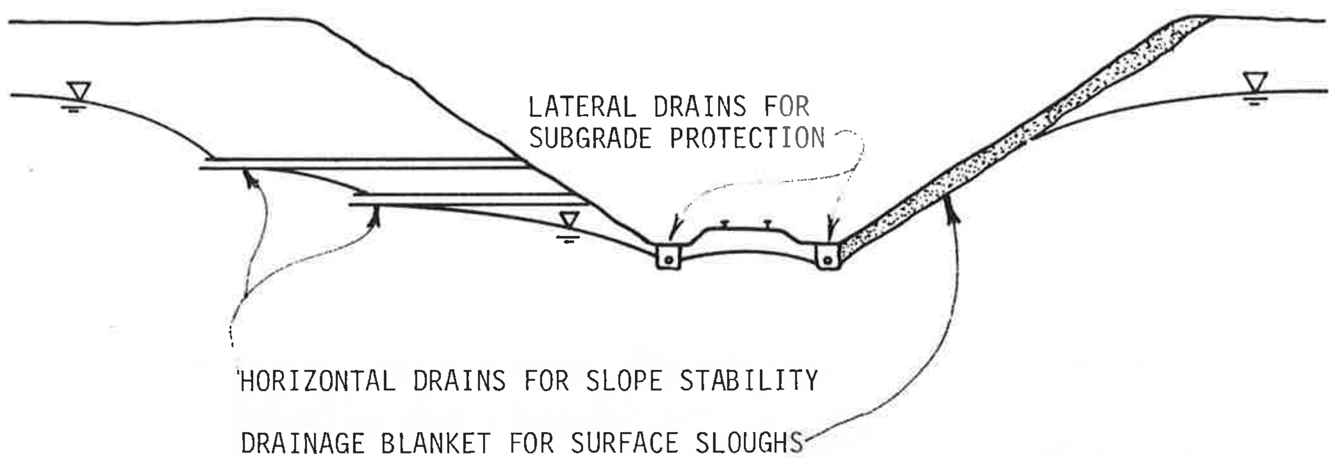
With proper design and installation, drainage can be used effectively for all soil types. Free-draining soils such as sands, gravels, and sand-gravel mixtures are easy to dewater. The high permeabilities of such soils mean that drainage structures may be widely spaced although water flows are likely to be great. Because of their high permeability and because these soils are not sensitive to disturbance, it may not be necessary or practical to provide drainage in these soils at great depths below the water table.

Drainage in fine-grained soils, such as silts or clays, is critical. Where excess water is present in these soils, soil strength may decrease to as little as 10 percent of its value when there is no excess water. These soils are also sensitive to disturbance by traffic and by cyclic loading, thereby reducing their strength even further. If a relatively low and stable subgrade moisture content can be maintained, a higher strength value can be used to design the track substructure.



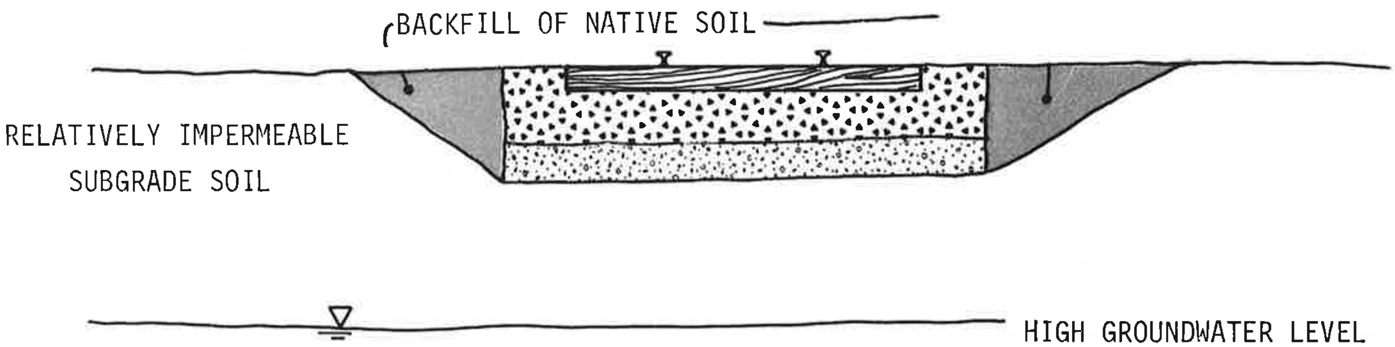
SURFACE BREAKOUT OF GROUNDWATER CAN CAUSE SURFACE SLOUGHS, SLOPE INSTABILITY, AND WET SUBGRADE CONDITIONS

(A)



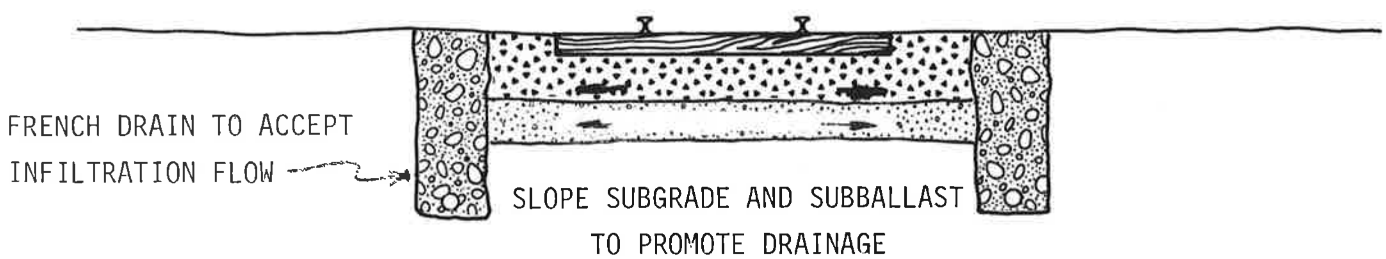
(B)

FIGURE 5-1. EXCAVATION BELOW GROUNDWATER LEVEL



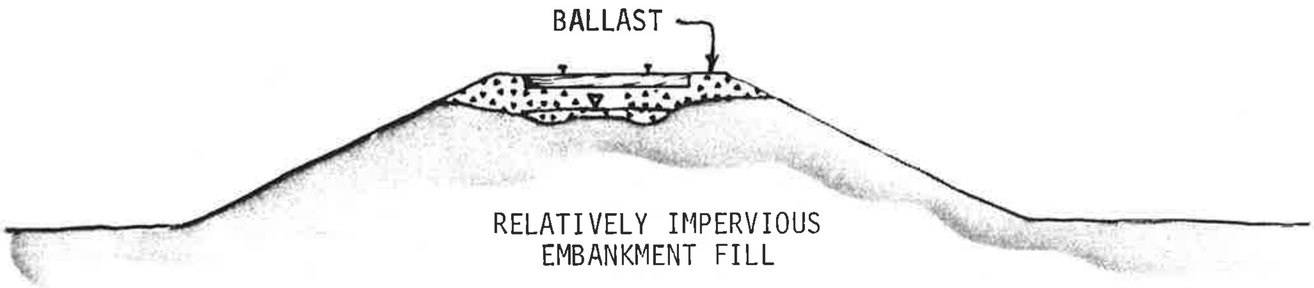
BECAUSE THERE IS NO WAY FOR BALLAST AND SUBBALLAST TO DRAIN, WATER WILL COLLECT AND CAN WEAKEN SUBGRADE. THIS WILL OCCUR EVEN THOUGH MAXIMUM GROUNDWATER LEVEL IS WELL BELOW THE TOP OF THE SUBGRADE

(A)

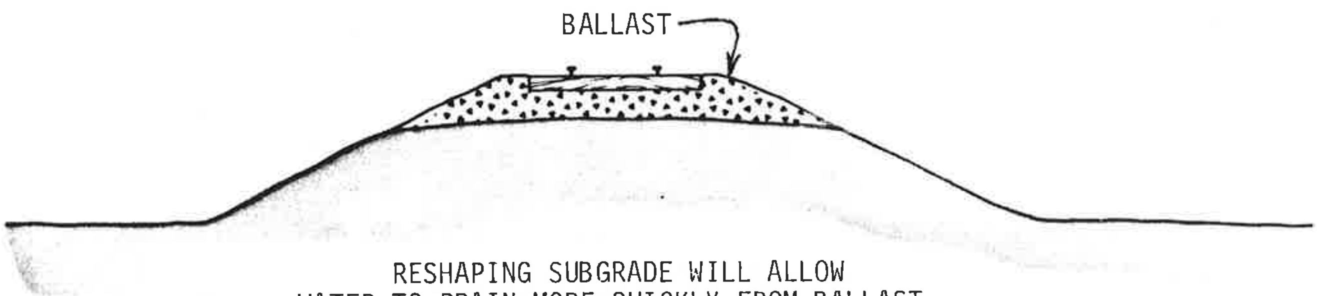


(B)

FIGURE 5-2. BATHTUB EFFECT



SLOPE OF SUBGRADE ALLOWS INFILTRATING WATER TO BE TRAPPED IN BALLAST AT SUBGRADE INTERFACE. WATER SOFTENING RESULTING IN SQUEEZE AND BALLAST POCKET FORMATION MAY OCCUR.



RESHAPING SUBGRADE WILL ALLOW
WATER TO DRAIN MORE QUICKLY FROM BALLAST

FIGURE 5-3. PUDDLING AT THE TOP OF AN EMBANKMENT

Engineering

Drainage below track has long been recognized as being a key factor in roadbed performance. General drainage criteria are available based on drainage measures that have yielded adequately performing roadbed. Examples of general criteria are given in the 1978 AREA Manual for Railway Engineering, Section 1.2.4 and by F. L. Peckover in his draft chapter submitted for the 1978 edition of Track Cyclopedia entitled "Roadway Stabilization and Drainage." Both sources recommend that the groundwater level should be maintained at least 4 feet below the top of the subgrade. Side drains installed using these criteria may maintain or improve performance of track roadbed by depressing the groundwater level, by allowing infiltrating water to drain away quickly, and by reducing the level of capillary rise in frost-susceptible soils. Additional data on capillary effects in frost heaving are provided in Section 5.3. The significance of these criteria may be evaluated by considering flow criteria as discussed in 1968 by H.R. Cedergren in Seepage, Drainage, and Flow Nets. If it is desired to maintain the water level 4 feet below the track, drains at the side of the track must be more than 4 feet below the top of subgrade to account for rise in water level away from the drains. Assume that 2 inches of precipitation per day enters the subgrade. If the subgrade is sand, permeability equal to 0.01 cm per second, the drains beside single track lines must be about 5 to 6 feet below the top of subgrade. However, if the subgrade is silt or clay, permeability equal to 0.0004 cm per second, lateral drains must be installed at least 10 feet below the top of subgrade at the center of the rails. Therefore, unless the track is raised on an embankment, it is impossible to satisfy this criterion without some underdrain beneath the ties. If double track or wider is considered, either deeper drains or intermediate drains between the tracks are required.

Explorations prior to drainage system design should define the soil, groundwater and climatic conditions. Characteristics of subgrade soils must be known including type, layering, and thicknesses. In existing track, the explorations have to be more detailed close to the surface to assess local variations over depth and laterally that may significantly affect the proposed drainage system. In particular, the lateral slope of the subgrade, the presence and depth of squeezes or ballast pockets, and the extent of ballast fouling will all have a marked effect on drainage system performance. As shown in Figures 5-2 and 5-3, installing a deep drainage system without considering the characteristics of ballast and subballast drainage may result in little change in substructure performance.

The depth to the groundwater table affects groundwater-lowering requirements, drainage measures for control of capillary rise, and the provisions of outlets for disposal of surface water. In assessing the groundwater conditions, the yearly high groundwater level which normally occurs in the spring is of concern. Explorations should be timed to coincide with yearly expected high groundwater levels. Observation wells should be installed to monitor the variations in water levels over at least the critical period of time.

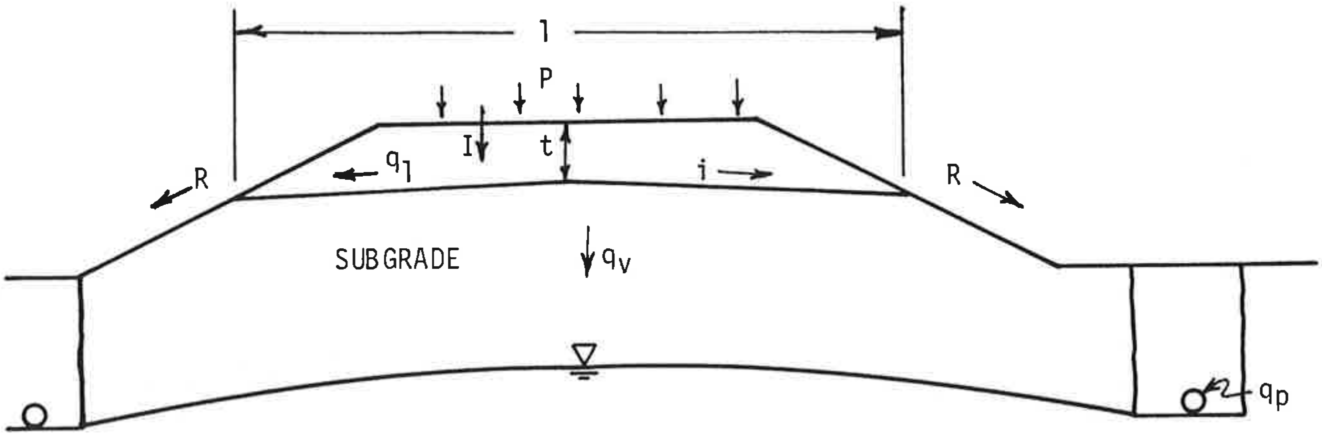
Typically, evaluation of soil permeability for drainage system design is based on correlations with soil classification and grain size distribution as contained in Section A8 of this report. The principal exception to this is where permanent groundwater lowering is required in permeable soils, i.e., clean sands and gravel. In this case, the quantity of flow can be substantial, and the cost of providing excess capacity is greater than the cost of permeability testing. Subgrade permeability testing is described in Section 2.3. However, if a significant drainage system design is contemplated, full scale pumping tests are the superior method of measuring in-situ permeability; description of pumping tests is beyond the scope of this report.

Soil shear strength and compressibility influence design of drainage systems in two ways. First, the sensitivity of soil mechanical properties (shear strength and resilience) to average and variations in soil moisture content influences the decision on whether and what type of drainage system is required to maintain or improve track performance. Second, slope failures and lateral creep affect drainage courses. Consolidation settlements may alter the grading of pipes and channels which determines their flow capacity. Subgrade movements due to consolidation or shear failure lead to development of ballast pockets or troughs in the subgrade (illustrated in Figure 5-3) that will impede gravity drainage of water at the subgrade surface.

Analytic methods can provide a rational basis for the design of subsurface and surface drainage systems. Drainage for track must be considered as a problem of inflow and outflow. The primary sources of moisture to the subgrade are precipitation, groundwater, and capillary action. Outflow is composed of surface runoff, lateral drainage, vertical drainage, drainage system discharge, and evaporation. Evaporation is typically a minor component of the total outflow except in arid conditions. An effective drainage system is one that will remove all excess water from the track substructure before the water deteriorates track performance.

In Drainage of Highway and Airfield Pavements (1974), H. R. Cedergren gives rational methods for design of pavement and other drainage systems. These methods can be used to design subsurface drainage systems for railroad roadbeds. Figure 5-4 illustrates some key factors that must be considered. Using even small values for infiltration, a wet subgrade will result unless the subgrade has relatively high vertical permeability or the surface drainage layer (i.e., the ballast and subballast) has very high horizontal permeability. In silt and clay -- where good drainage is most critical -- the vertical permeability is very small, yielding very little if any vertical infiltration through the subgrade; that leaves lateral drainage as the only means of draining infiltrating water.

Analysis of groundwater flow can be made based on mathematical solutions that require evaluation of the permeability coefficient of the subgrade soils. Analytic solutions may be found in the U.S. Army, Corps of Engineers technical manual on Dewatering and Groundwater Control for Deep Excavations (1971),



P = PRECIPITATION (RAIN OR SNOW MELT)

R = DIRECT RUNOFF

I = INFILTRATION OF PERVIOUS STONE BALLAST ($I \approx P$)

q_l = LATERAL COMPONENT OF FLOW RESULTING FROM INFILTRATION

$q_l = K_b \cdot i \cdot t$ (PER FOOT OF TRACK)

K_b = PERMEABILITY OF BALLAST OR SUBBALLAST
i = GRADIENT OR SLOPE OF SUBSTRUCTURE SURFACE (MINIMUM OF 2%)
t = THICKNESS OF FLOW LAYER

q_v = VERTICAL COMPONENT OF FLOW RESULTING FROM INFILTRATION

$q_l = K_s \cdot i \cdot l$ (PER FOOT OF TRACK)

K_s = VERTICAL PERMEABILITY OF SUBGRADE SOILS
i = GRADIENT ($i = 1.0 \rightarrow$ GRAVITY DRAINAGE)
l = SUBGRADE INTERFACE WIDTH

q_p = COMPONENT OF OUTFLOW IN PIPE

$q_p = q_v + q$ GROUNDWATER

FIGURE 5-4. GENERAL BASIS OF DRAINAGE DESIGN the 1968 text by H. R. Cedergren entitled Seepage, Drainage, and Flow Nets, or the chapter written by C. I. Mansur and R. I. Kaufman in Foundation Engineering (1962), G. A. Leonards, editor. Designs should be based on maximum anticipated flows, as determined from the highest expected groundwater level.

Description: Drainage Geometry

The two basic design factors for a drainage system are establishment of drainage geometry and selection of the type of drainage structure. Selection of geometry is based on evaluation of track geometry, surrounding ground topography, location of natural drainage courses, and groundwater levels. The types of drain geometries are discussed below.

Lateral Drains

Lateral drains, also known as side drains, are located on one or both sides of a track, paralleling its route until an outfall is reached. These are probably the most common of all railroad drain installations and can be designed to intercept and carry surface runoff, seepage, and groundwater flows.

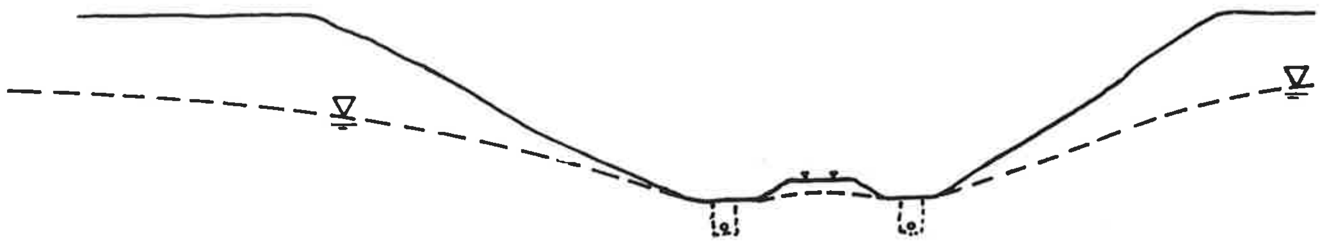
Figure 5-5 illustrates several different cases where lateral drains may be used. In some cases, the lateral drains may be two separate installations. Surface ditches transmit precipitation runoff, while buried pipe drains carry groundwater flows. Buried lateral drains may be of particular value for grade crossings and special track in wet areas. However, providing an outlet for these drains may be difficult in areas where topographic relief is slight. Normally, lateral drains end and outlet near the edge of embankment sections or at natural drainage courses. Natural drainage courses control the geometric design of drainage systems.

Offtake Drains

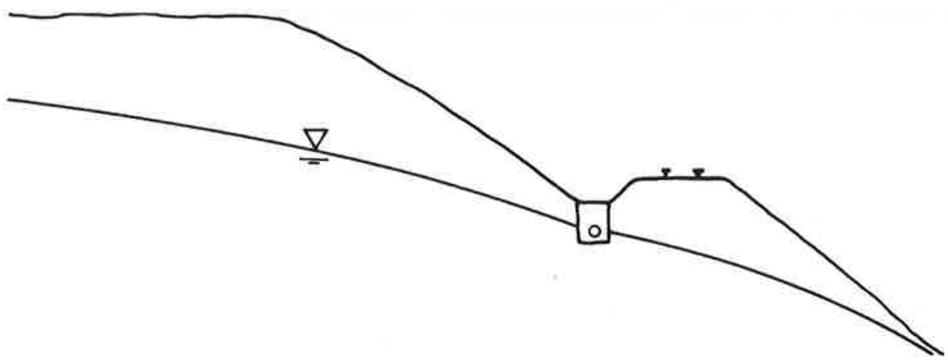
Offtake drains provide an intermediate outlet for lateral drain systems to limit lateral drain length or to satisfy slope limitations. Offtake drains intersect lateral drains and provide a shorter distance to a natural drainage course than would be available parallel to the track.

Interceptor Drains

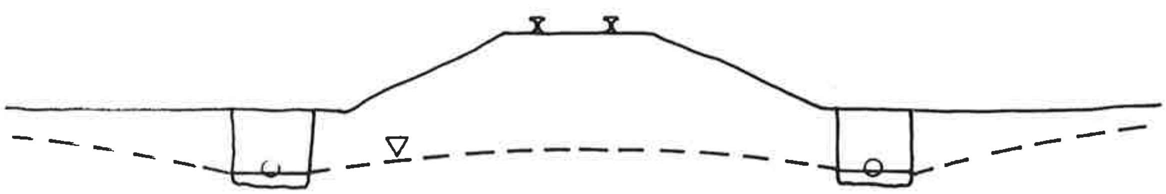
Interceptor drains are installed at the top of cut slopes to intercept surface runoff and, in some cases, groundwater flow. Intercepting runoff before it can reach the slope surface reduces erosion on the slope and the tendency for surface sloughing. In deep cuts, intermediate interceptor drains may be installed on the slope to gather seepage and runoff not caught by the uppermost drains. Figure 5-6 illustrates cases where interceptors might be used in a cut slope.



(A) DITCHES TO CARRY RUNOFF AND GROUNDWATER FLOW IN CUT SECTION



(B) LATERAL DRAIN FOR RUNOFF AND GROUNDWATER CONTROL IN SIDE HILL CUT



(C) LATERAL DRAINS FOR CONTROL OF GROUNDWATER BELOW EMBANKMENT

FIGURE 5-5. APPLICATIONS OF LATERAL DRAINS

In some instances, it may be necessary to divert flow from the interceptor drain to the base of the slope to limit interceptor capacity or because of physical barriers. Such diversions should be made in a controlled manner, with the flow carried by a paved flume or a pipe, to avoid erosion of the slope. Special care should be taken at the outlet of the diversion channel because severe erosion can occur due to high flow velocities running down the slope.

These drains also can be used to intercept groundwater flows that might otherwise emerge as seepage on cut slopes. When uncontrolled, this groundwater flow may cause deep slope instability, surface sloughing, and erosion. When interceptors are used for groundwater control, they must extend quite deep to be effective. Figure 5-6 shows how interceptors can be used to control groundwater.

Cross Drains

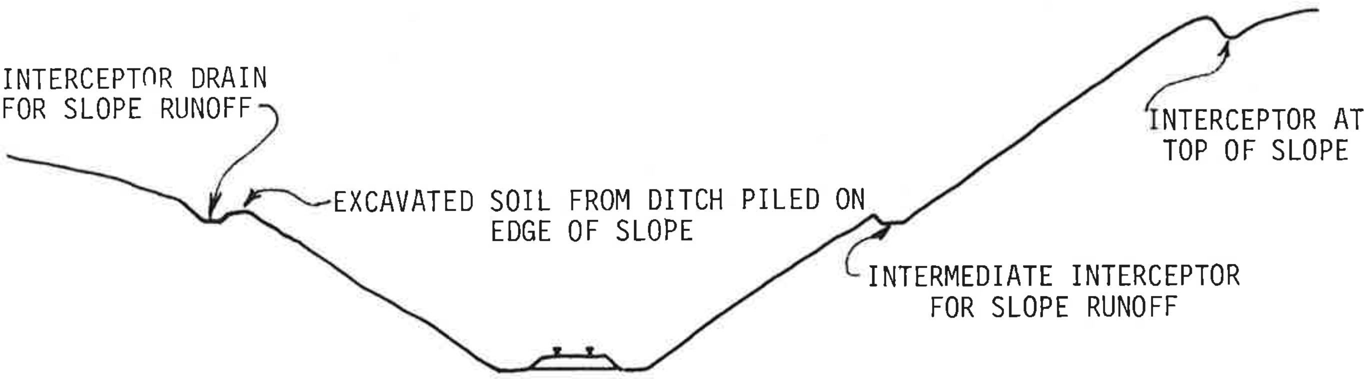
Cross drains run beneath tracks. These drains supplement lateral drains when the spacing of lateral drains is insufficient to control groundwater at the center of the track. Because the drains run under tracks, pipes or French drains must be used. Cross drains are most commonly used in yards or areas where there are several parallel tracks. Although the design procedures presented earlier can be used to design these drains, most of the drains installed have not been rigorously designed. Peckover (1978) states that most of these drains are from 2 feet to 10 feet deep and discharge into the lateral drainage systems. Strict criteria are recommended for cross drain design since the drains are more difficult and more expensive to replace in the event of a failure and would require disruption of the track. Figure 5-7 illustrates a typical cross drain system geometry.

Horizontal Drains

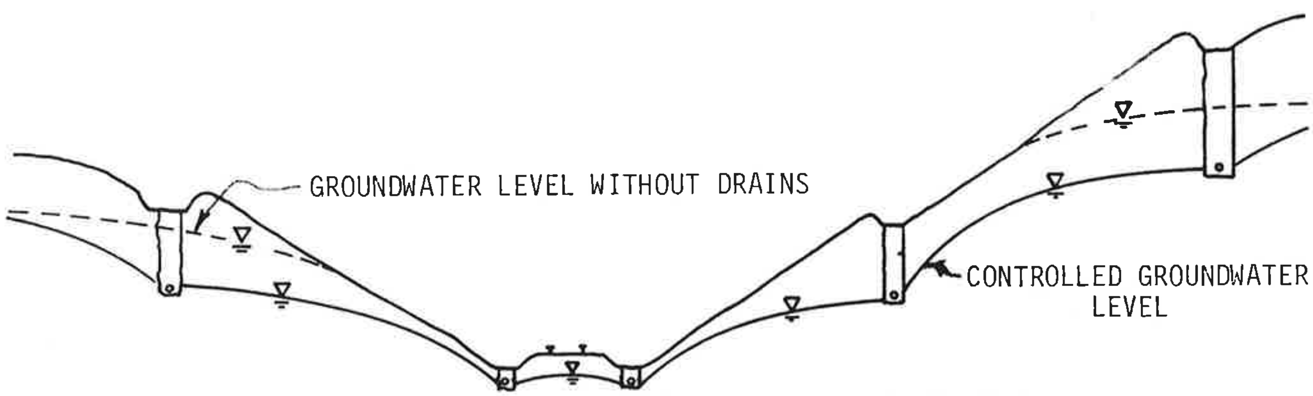
Horizonal drains are drilled into cut slopes to provide a conduit for groundwater to emerge. These drains may be used in place of or to supplement interceptor drains and are expected only to transmit groundwater flows. Most experience with horizontal drains has been in stabilizing highway cut slopes, but it is directly applicable to railroad situations. Discharge of the horizontal drains on the slope surface must be managed to prevent erosion. Figure 5-8 illustrates how horizontal drains can be used.

Description: Types of Drainage Structures

There are three principal types of drainage structures used in railroad substructure: ditches, pipes, and French drains. These structures can be arranged in a variety of geometries, as described above. In addition to special drainage structures, track drainage may be provided or improved simply by raising the track.

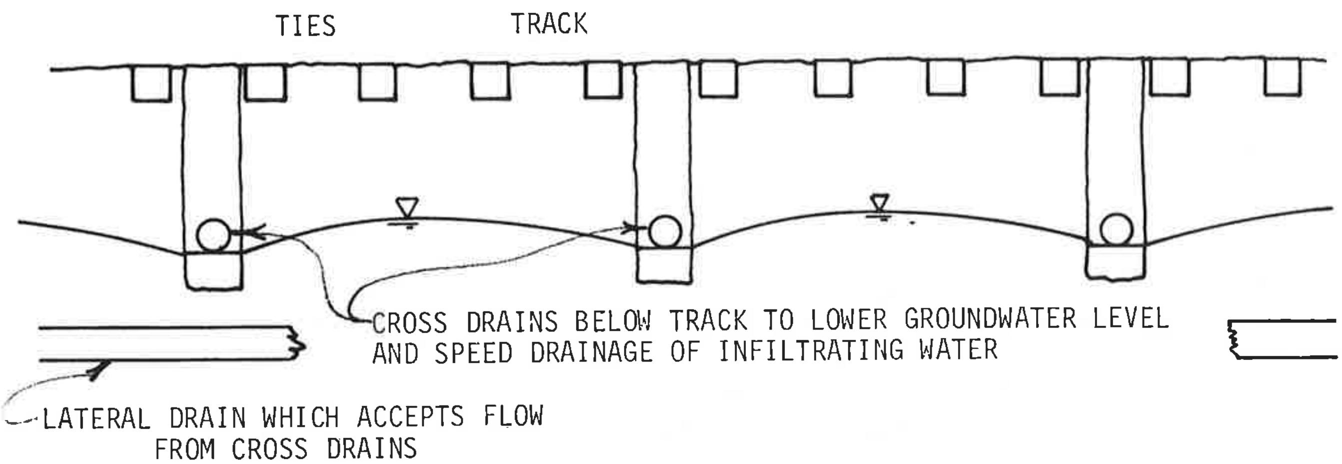


(A) INTERCEPTOR DRAINS FOR SLOPE RUNOFF

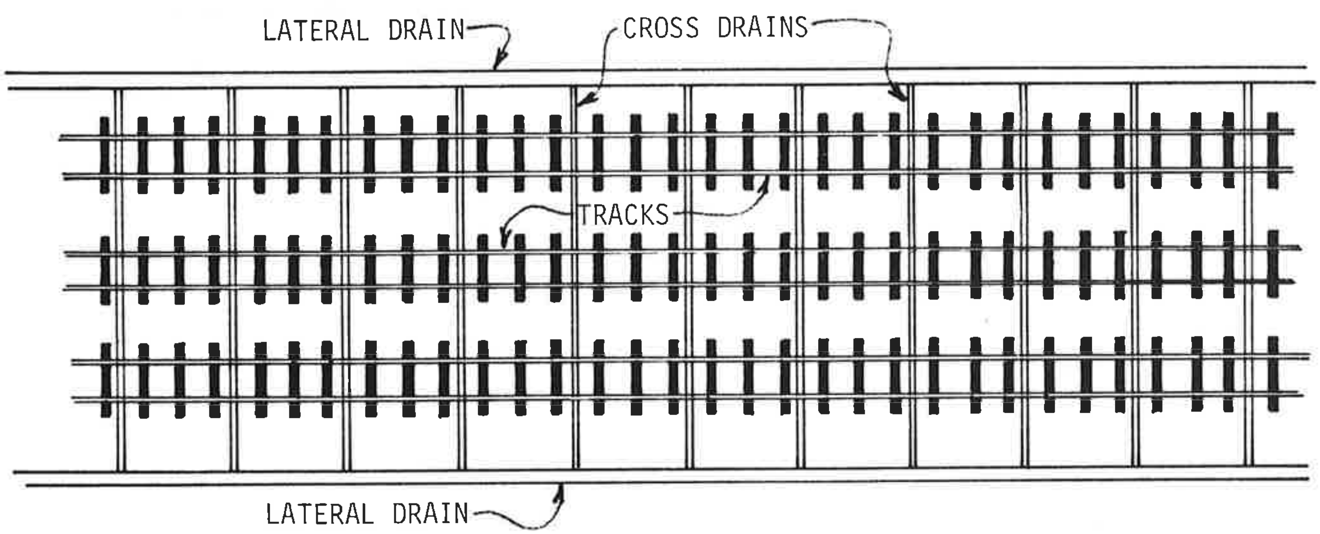


(B) COMBINED INTERCEPTOR DRAIN SYSTEM FOR SLOPE RUNOFF AND GROUNDWATER CONTROL

FIGURE 5-6. INTERCEPTOR DRAINS



(A)



(B)

FIGURE 5-7. CROSS DRAINS

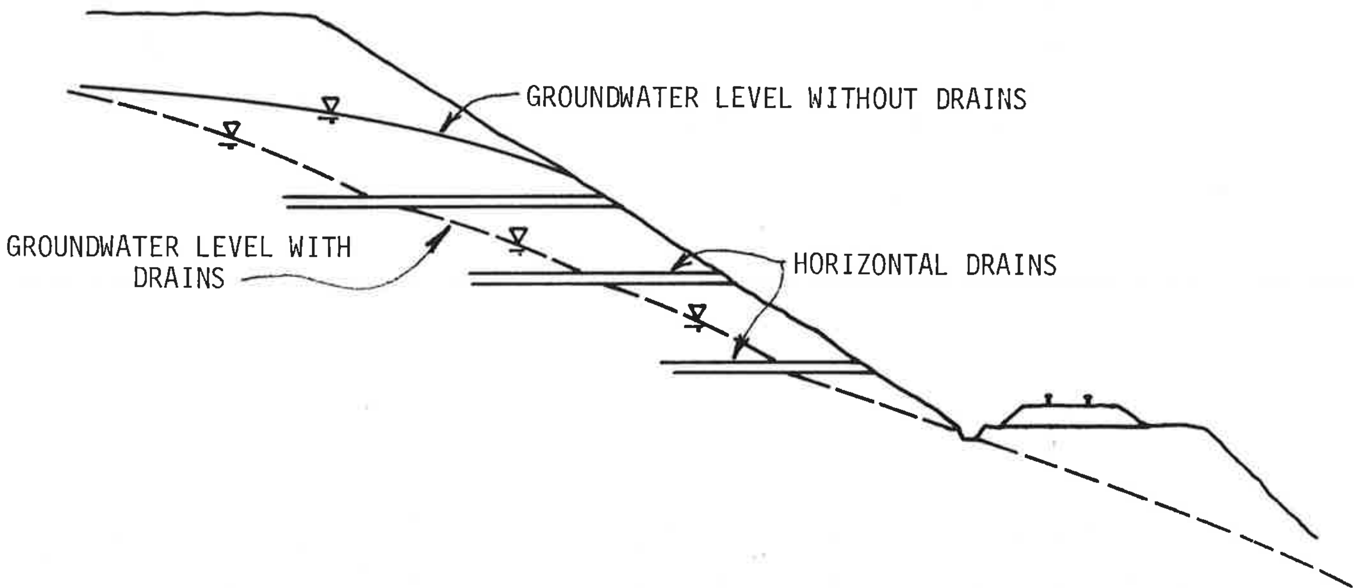


FIGURE 5-8. HORIZONTAL DRAINS

Ditch Drains

Application - Ditch drains are primarily used to carry surface runoff flows. Ditches are the most economical way to provide the high flow capacity required to remove storm runoff, and can be used for lateral, offtake, or interceptor drains. Depending upon soil conditions, groundwater conditions, and depth of the drain, open ditches may also provide groundwater control. Because of their ease of construction and maintenance, ditch drains are the most common type of drains used in railroad practice.

Description - Ditch drains can be V-shaped or trapezoidal-shaped (flat bottom, sloping sides). Trapezoidal is preferred because it allows for easier maintenance and because some debris can collect in the bottom of the ditch without blocking the flow completely. Section 1.2.4 of the AREA Manual recommends a minimum bottom width of 3 feet. For both the V-shaped and trapezoidal ditches, the side slopes should not be so steep as to be unstable or promote erosion. The ditch must not undermine the stability of cut slopes or track embankments. Where ditches are used as interceptors for surface flow at the top of cut slopes, it is often desirable to line the ditch (e.g., with asphalt or Portland cement concrete or a membrane) to reduce the amount of water infiltrating into the slope. If the grade of the ditch becomes steep, a lining will be needed to limit erosion.

Engineering - Drainage ditch design involves evaluating the quantity of inflow and the flow capacity of the section. Hydrologic analyses can be performed to predict runoff amounts and seepage flows. The basic features controlling flow along the ditch are gradient, shape, and cross-sectional area. Frequently, the shape and cross-sectional area of the ditch are controlled more by ease of maintenance or other factors than by flow requirements.

Ditches must be sloped steeply enough to prevent sedimentation but not so steeply as to encourage erosion of the ditches. The minimum gradient required to prevent sedimentation is approximately 0.25 percent, as recommended in the AREA Manual. Flow velocity should be kept below the erosion limits indicated in Table 5-1. If higher velocities are anticipated, the ditch may be protected from erosion by pavement, crushed stone lining, erosion stabilization mats, Gabions (wire baskets filled with stones), or other measures. Dams may be erected across the ditch to reduce flow velocities; however, these will reduce the effectiveness of the ditch to provide drainage. Offtake ditches may be required in areas where the drain gradient is not sufficient to carry the flow over a greater distance or to increase the flow velocity where sedimentation is a problem.

TABLE 5-1. LIMITING FLOW VELOCITIES TO PREVENT EROSION

Material at Bottom of Ditch	Flow Velocity (feet per second)
Sand	up to 2
Loam (Sandy Silt)	2 to 3
Grass	2 to 3
Clay	3 to 5
Clay and Gravel	4 to 5
Good sod, coarse gravel, cobbles, soft shale	4 to 6

Cost - Ditches are relatively easy and inexpensive to construct. A wide variety of equipment can be used to construct ditches and perform similar excavation work. Besides conventional earthwork equipment, some equipment has been developed specifically for use in railroad work, including spreader-ditchers to shape and clean lateral ditches. The ditcher is pushed by a locomotive and can be used to cut and shape ditches and berms, and to cut slopes at the side of the track. Ditches can be constructed and maintained using machinery with little need for hand labor.

The cost of constructing a ditch will depend largely on the specifics of working conditions. If large equipment can be used, the cost may be approximately \$1.00 to \$3.00 per cubic yard of material excavated. In more restricted work areas, the cost of excavation may be \$5.00 or more per cubic yard. This does not include the cost of seeding, sodding, or other measures that may be required for erosion protection. An approximate cost for installing a ditch drain is \$2.00 to \$4.00 per lineal foot. Erosion protection will add approximately \$1.00 to \$15.00 per square yard to the cost of the ditch, depending on the type of lining (1).

Discussion - After construction, open trenches can be observed during or after rainfalls to assess their drainage performance. If widening or deepening is required, these actions can be performed at low cost. Construction and maintenance costs of open ditches are low because the operation is largely mechanized. However, if erosion protection is required, the cost-per-foot is several times greater than that for an unlined ditch.

Pipe Drains

Application - In all types of soils, pipe drains are used to intercept and carry surface runoff or groundwater flows, and are installed when there is insufficient space to construct an open ditch drain at the desired level. Pipes are used to carry flow beneath tracks, such as for offtakes of lateral drains where the natural drainage outlet is only available on one side of

(1) Cost information is included to permit general conparisons. Because it can be variable with time and location, specific studies will require updated cost information.

the track. Pipes also may be used in place of a lined ditch in applications such as an offtake for a slope interceptor drain.

Description - Pipe drains consist of a pipe surrounded by a permeable filter material that acts as a collection zone. Figure 5-9 illustrates a pipe drain installation. The pipe may be slotted, perforated, open-joint, or porous so that water can enter the pipe and be transmitted along its length while keeping soil particles out. Corrugated steel, asbestos cement, porous concrete, and plastic are common drain pipe materials. Clay pipes are used much less frequently now than in previous years.

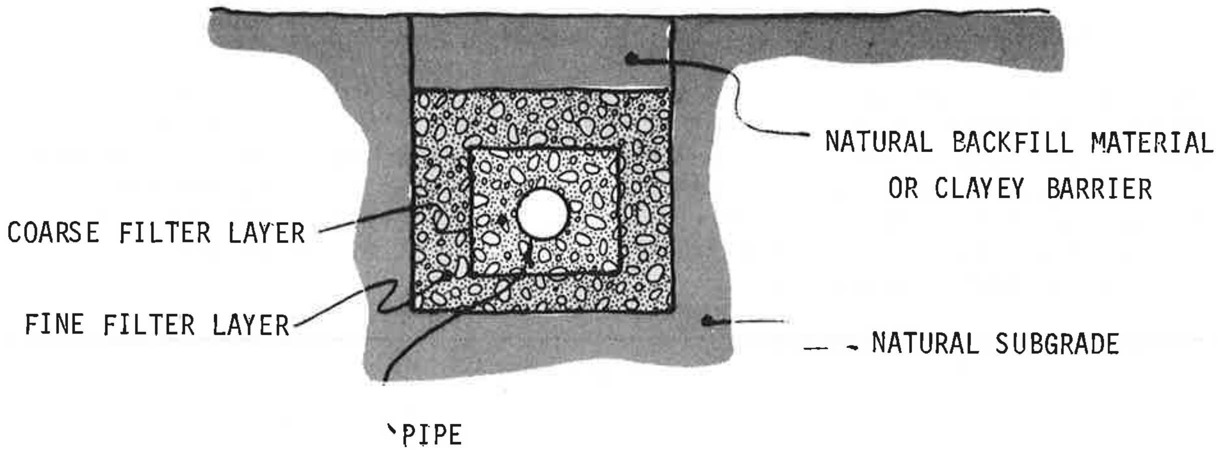
The basic installation of a pipe drain requires excavation of a trench, installation of filter layer(s) below the pipe, installation of the pipe, and backfill with the filter layers and general backfill. Two critical concerns in this installation are: (1) to place the filter layers in proper thickness and arrangement to perform their functions, and (2) to obtain or manufacture granular materials that satisfy the filter criteria. Filter criteria are discussed below under Engineering.

Rather than granular filters, filter fabrics have been used increasingly in underground pipe drain installations. Filter fabric is a plastic cloth that is permeable to water yet prevents the passage of soil particles. (Fabrics are discussed more fully in Section 5.3). With the fabric lining the walls and top of the trench, the filter requirements described above are not necessary, and the pervious collection zone aggregate surrounding the pipe can be coarse, crushed stone or other easily obtainable porous material. It is also easier to see that the fabric is installed in its proper place than to check proper installation of granular filter zones.

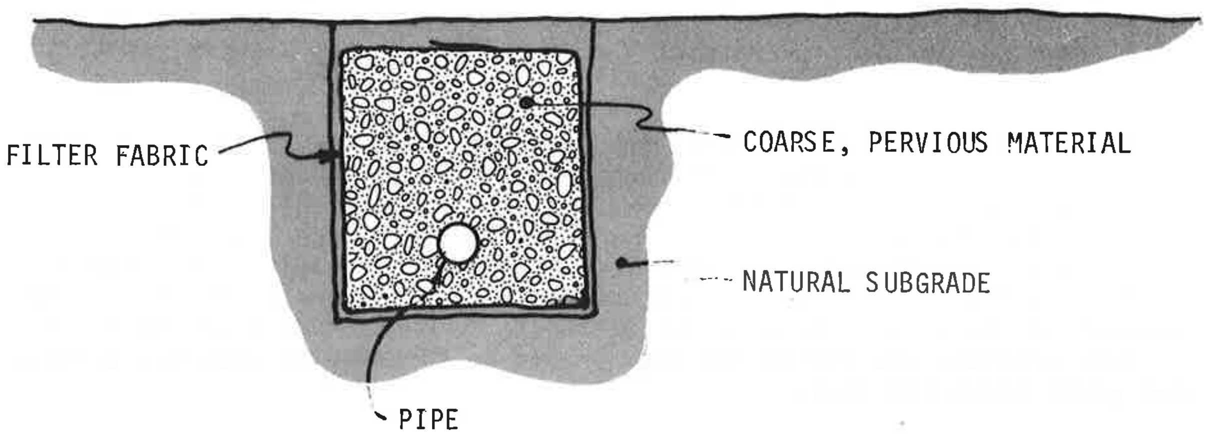
If the pipe drainage system is designed to carry only groundwater flow, the near-surface trench backfill should be a low-permeability soil. A lined ditch drain may be constructed over the pipe drain to carry surface water.

Drain pipe sizes are based on required flow capacity. Pipe sizes range from 50mm (2 inches) to 600mm (24 inches) in diameter. Larger sizes are available as interceptors to transmit flow. However, a 150mm (6-inch) minimum size is recommended to permit cleanout in the event of blockage. If long lengths of pipe drain are required, it is recommended that manholes be provided at regular intervals, usually not more than 500 feet, to permit inspection and cleanout of the pipe. The pipe outlet should be unobstructed and designed to limit erosion; the end of the pipe should be screened to keep out animals that might block the flow.

Engineering - The amount of flow in the pipe drain is estimated from analysis of groundwater flow or general experience. If surface water is to be carried by the pipe, this flow must be included. Flow along the pipe is analyzed based on standard hydraulic engineering procedures. The pipe size required for groundwater control applications is frequently determined by maintenance/cleanout requirements rather than by the flow to be carried.



(A)



(B)

FIGURE 5-9. PIPE DRAINS

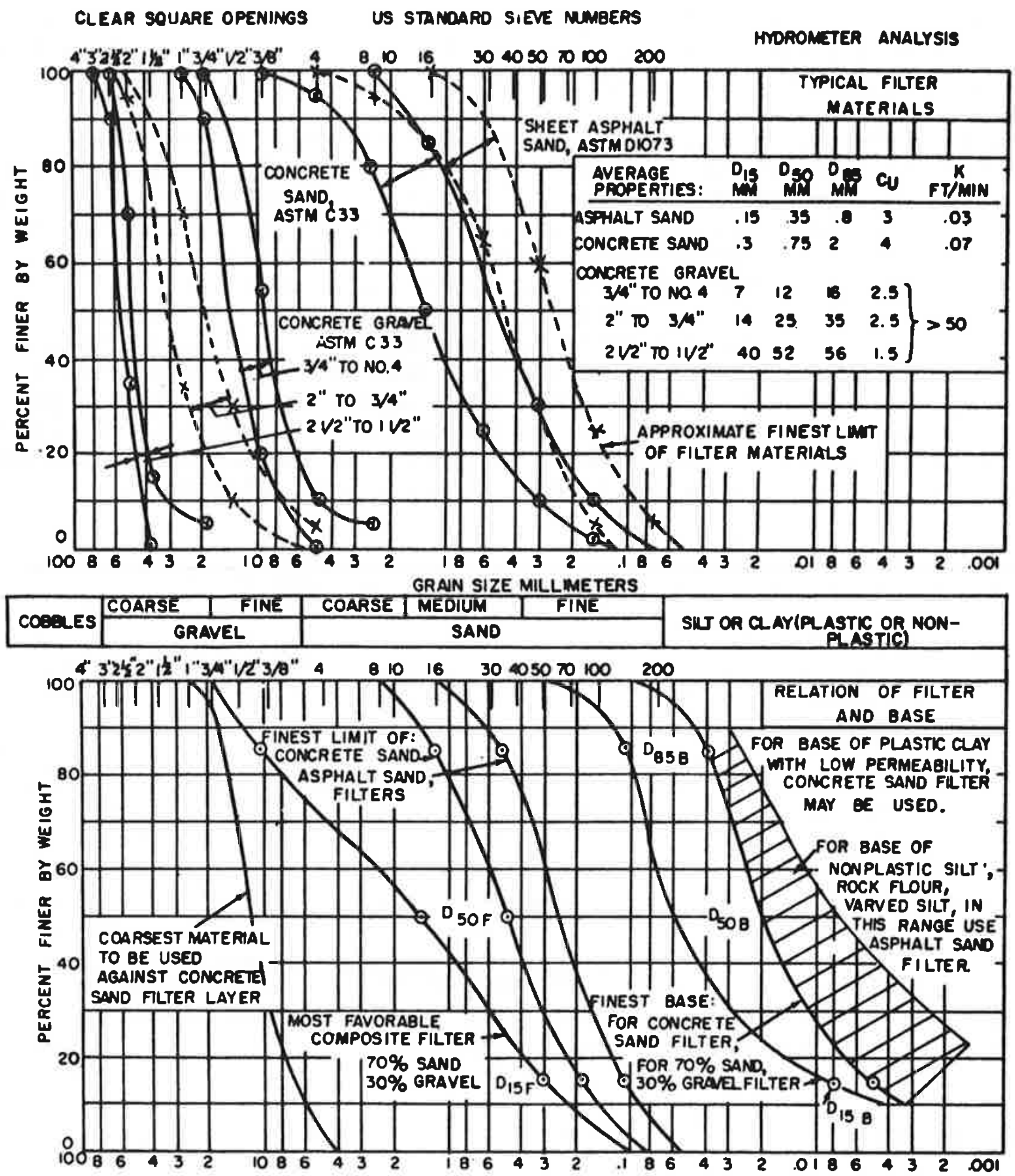
Sedimentation in pipes is also a concern. Peckover recommends a minimum gradient of 0.15 percent, whereas the AREA Manual recommends a minimum flow velocity of 0.6m (2 feet) per second to limit sedimentation.

Special engineering and construction consideration must be given to the filter requirements for the pipe. Filters prevent the movement of fine soil particles while permitting the flow of water into more permeable collection zone materials. Soil particles can clog the pores of the backfill collection zone, resulting in an ineffective drainage system; they also can clog the pipes. Filters can be constructed of natural granular materials, or manmade filter fabrics can be used. Figure 5-9 illustrates both of these cases. Filters constructed of natural materials should be designed in accordance with the criteria presented in Figure 5-10. Several filter layers may be needed to prevent movement of particles and clogging of pores. When pipe drains are installed below track, as in the case of cross drains, all backfill for pipe drains should be compacted to 95 percent to 100 percent of the maximum dry density as determined by the Standard Proctor test (ASTM D698).

Cost - The cost of a pipe drain system is comprised of the costs of: excavating a trench; the pipe, backfill, and filter materials; manholes; and backfilling the trench. The cost of pipe in the 6-inch to 12-inch size varies from about $1.05 per lineal foot to $5.00 per lineal foot, depending on the pipe material and size. Excavation costs are about $2.00 to $3.00 per cubic yard of material excavated. (This does not include trucking costs.) Gravel or stone backfill costs approximately $6.00 to $10.00 per cubic yard for hauls of two miles or less. Properly graded filter materials may cost more than $10.00 per cubic yard. Manholes at 500-foot spacings will add approximately $0.75 to $1.00 per lineal foot to the cost. If filter fabric is used, it will cost approximately $0.10 per square foot of fabric to line the trench and overlap at the top. However, graded filter materials can be eliminated if filter fabric is used. The approximate cost of installing a 6-inch pipe drain in a 6-foot-deep by 3-foot-wide trench would be approximately $15.00 to $20.00 per lineal foot. In 1980, R. E. Ahlf reported in "Matching M/W Practice to Required Use of Track - the Costs," that an 8-inch pipe drain beneath track cost about $31.50 per foot of drain.

French Drains

French drains are trenches filled with stone. The stone both collects the water and transmits the flow through its voids. The principal difficulty with French drain installations is that fine soil particles enter the voids, clog the pores, reduce the permeability, and destroy the effectiveness of the drain. Graded granular filters could be placed around the stone to prevent contamination; however, this would be expensive and difficult to construct. With the adoption of filter fabric, practical, permanent French drains can be installed, as shown in Figure 5-11. Filter fabric permits installation



General requirements:

1. To avoid head loss in filter: $\frac{D_{15F}}{D_{15B}} > 4$, and permeability of filter must be large enough to suffice for the particular drainage system.
2. To avoid movement of particles from base: $\frac{D_{15F}}{D_{85B}} < 5$, $\frac{D_{50F}}{D_{50B}} < 25$, $\frac{D_{15F}}{D_{15B}} < 20$

   For very uniform base material ($C_u < 1.5$): D_{15F}/D_{85D} may be increased to 6

   For broadly graded base material ($C_u > 4$): D_{15F}/D_{15B} may be increased to 40
3. To avoid movement of filter in drain pipe perforations or joints:

   D_{85F}/slot width > (1.2 to 1.4) D_{85F}/hole diameter > (1.0 to 1.2)
4. To avoid segregation filter should contain no sizes larger than 3".
5. To avoid internal movement of fines, filter should have no more than 5% passing No. 200 sieve.

Reproduced from Soil Mechanics, Foundations, and Earth Structures, p. 7-8-14, U.S. Navfac Design Manual DM-7. Year of first publication: 1971.

FIGURE 5-10. DESIGN CRITERIA FOR PROTECTIVE FILTERS

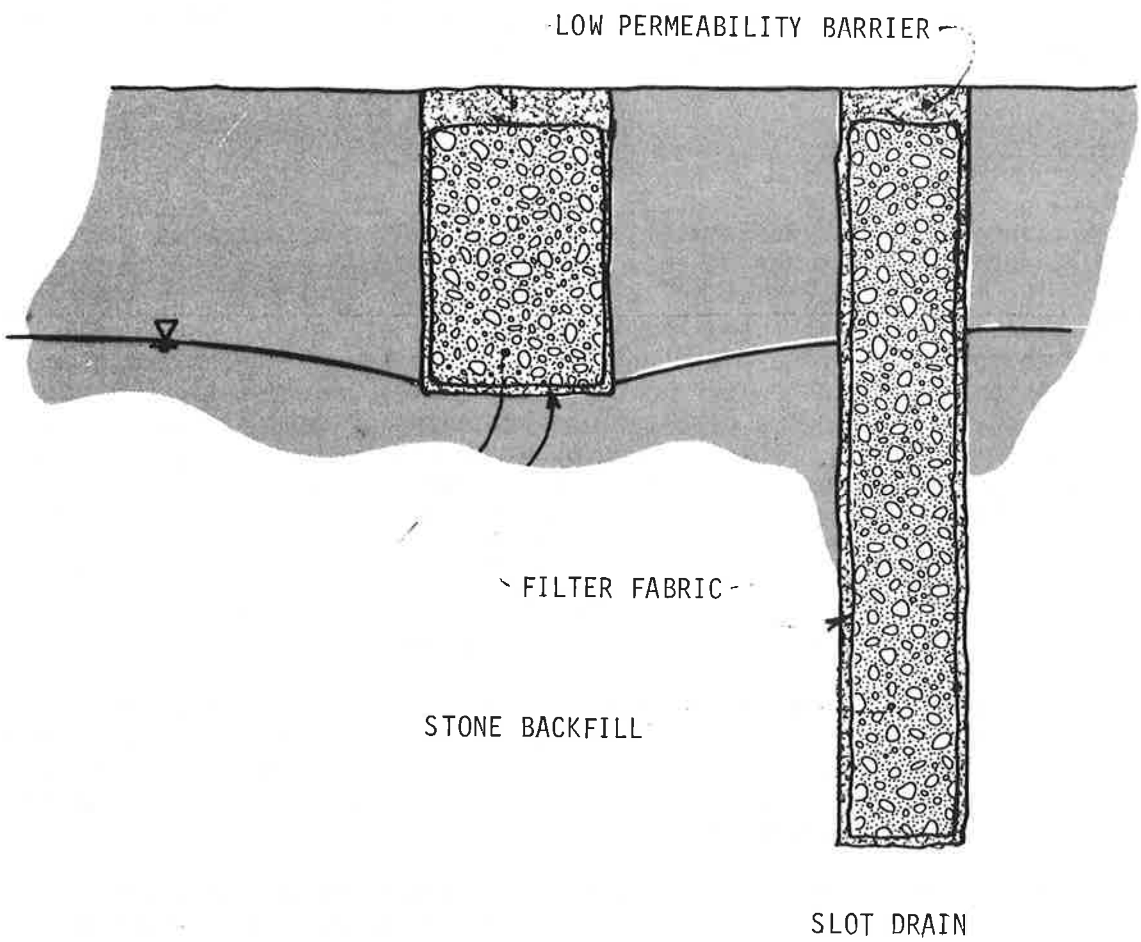


FIGURE 5-11. FRENCH DRAINS

of very narrow French drains. A 12-inch-wide trench might be excavated in a slightly cohesive soil as much as 10 feet deep if the sides stand unsupported. The fabric and stone can be installed quickly, limiting the volume of earth removed and the volume of aggregate replaced.

The cost factors of French drains are similar to those of a pipe drain excluding the pipe. Correspondingly, flow capacity is less in a French drain than in a typical pipe drain. As mentioned previously, flow capacity is frequently not a critical factor in an underdrain system. A French drain may be practicable where flows are small; a pipe may be included in sections where greater flow is anticipated.

Special Techniques

Horizontal Drains - Horizontal drains typically are installed in slopes to arrest deep movements and shallow sloughing due to groundwater pressure. These drains typically consist of 2-inch perforated steel pipes or slotted plastic pipe installed at a shallow incline into a hillside. The length, spacing, and slope of these pipes will vary depending upon the slope geometry, soil conditions, and groundwater conditions. Design and installation of horizontal drains requires a detailed understanding of geologic and hydrologic conditions in the slope. As described by H. R. Cedergren in *Seepage, Drainage and Flow Nets* (1967), horizontal drains installed to stabilize highway cut slopes in California have varied in length from 50 feet to 300 feet and have been spaced at intervals ranging from 25 feet to 100 feet. The pipes are placed in drilled holes sloped at gradients of 3 percent to 20 percent. Varying one of these parameters affects the other two. Special horizontal drilling equipment is required for installation of the pipes.

To assess the drainage requirements behind a slope, installation of piezometers in the slope is required. A geohydrologic analysis of groundwater flows can then be conducted based on different drain spacings and lengths. This analysis should be coupled with a slope stability analysis to determine the influence of groundwater changes on slope factor-of-safety.

To some extent, a horizontal drain system should be designed and installed by a trial-and-error procedure. If initial design spacing and length do not meet drainage requirements, more pipes and/or longer pipes can be installed. The design parameters can then be varied until desired results are obtained.

Horizontal drains require periodic maintenance. Cleaning the drains is accomplished with the same equipment used in the installation by using a drill bit within the pipe. Water jetting through the casing under pressures of 150 psi to 200 psi has also been effective in cleaning drains.

Drain outlets must be placed so that slope erosion will not occur. In cold climates, outlets for drain pipes will freeze, shutting off the flow. In this case the outlets have to be protected by such means as providing an underground collection system and ground cover for insulation so that they operate even in freezing conditions.

Finally, slot or perforation sizes must be selected to prevent movement into the pipe of soil particles that might cause a void to develop around the pipe or clog the pipe. Criteria for pipe perforations are provided in Figure 5-10.

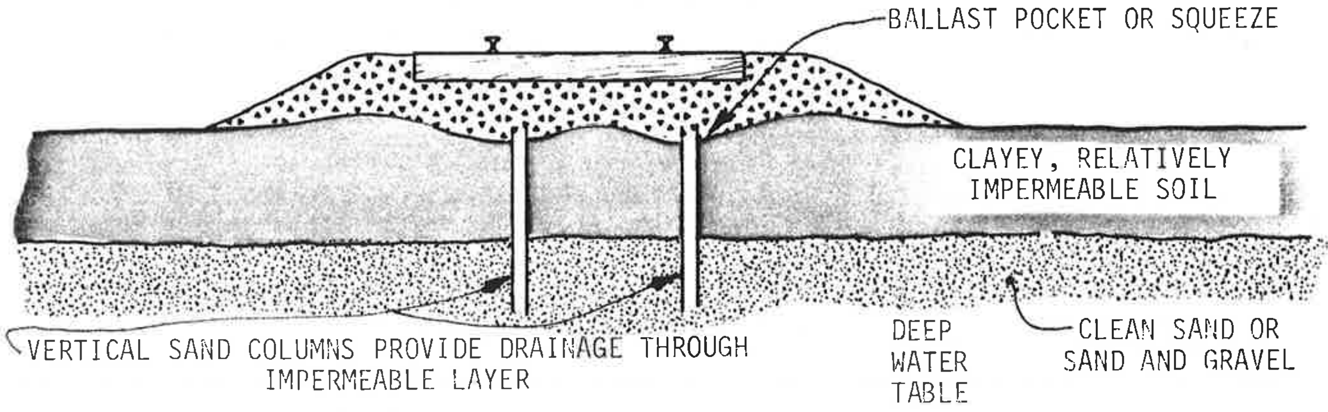
Sand Drains - Many existing tracks have ballast pockets and squeezes primarily in impermeable, silty and clayey subgrade soils. In cases where the groundwater level is well below track level, it may be possible to install vertical sand columns through the clay and into underlying pervious soils. These sand columns can provide a vertical conduit for water trapped in shallow subgrade pockets to percolate down into deeper permeable zones. Figure 5-12 illustrates how such a system might be laid out.

The design of sand drains involves calculation of the expected infiltration and the capacity of each individual drain. The sand drains would increase the amount of vertical seepage and would supplement lateral drainage of surface water. Related to the shallow sand drain technique is installation of sand-filled spud holes, described in Section 5.5. Based on experience with sand drains in foundation engineering, it is preferred to install the drains in an augered or jetted hole in order to limit disturbance of the cohesive soil subgrade. A hollow stem auger might be used if there is difficulty in keeping the augered hole open.

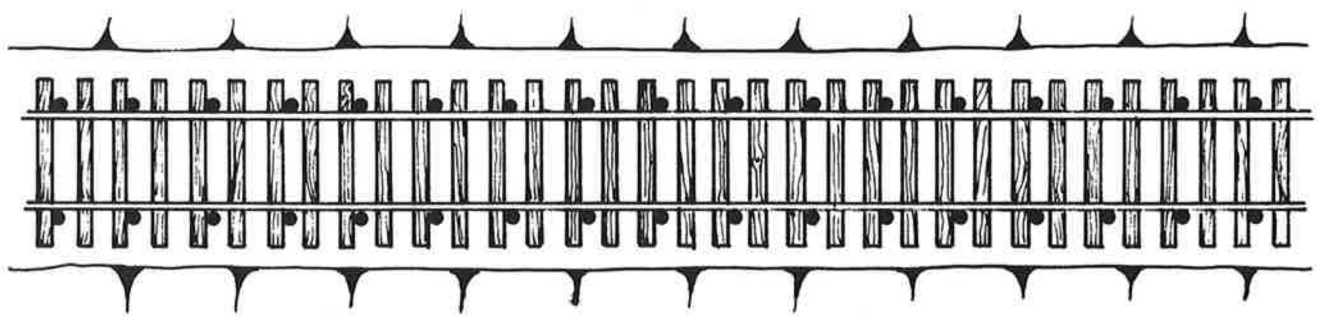
The bottom of Figure 5-12 illustrates a sand-filled drain installed laterally to conduct water from under the rails and into lateral drains. There are many difficulties in constructing such a system. First, the sand drain must intersect the bottom of a ballast pocket. Second, the drains will be subject to clogging unless the sand used in the drain satisfies the protective filter criteria of Figure 5-10. The drain is also subject to high dynamic stresses from the track. An installation of this type is not likely to remain effective for long periods of time.

Discussion

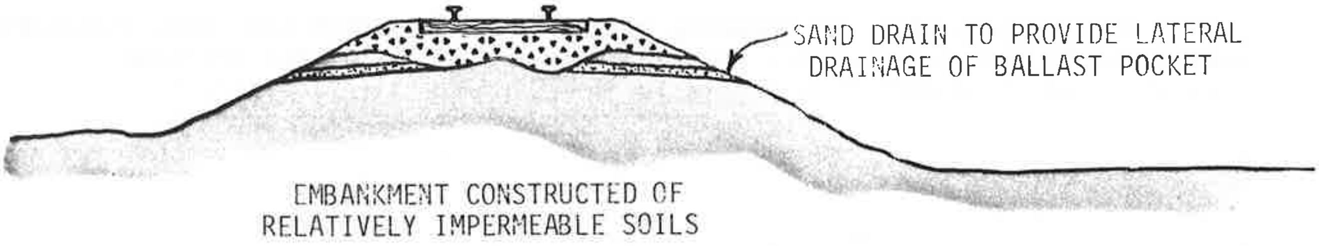
In the same way that the presence of excess water within the track structure contributes to most substructure problems, provision of adequate drainage of rainfall and groundwater generally is the most effective and efficient means of maintaining track performance and correcting subgrade problems. The mechanical performance of all elements of the track substructure deteriorate if the materials become saturated with water, as discussed in Sections 2 and 3. Improvement in drainage of water from the track should be considered in all new track construction planning and in planning all track maintenance and rehabilitation programs. The combined influence of improved drainage,



(A) SECTION OF VERTICAL SAND DRAIN INSTALLATION



(B) PLAN OF SAND DRAIN INSTALLATION



(C) LATERAL SAND DRAIN INSTALLATION

FIGURE 5-12. SAND DRAINS

together with some other stabilization methods, will typically provide an optimum treatment of a wide range of track problems.

Each of the drainage structure types discussed has a range of applications in which it is the preferable choice. In an overall drainage plan, a combination of these structures will provide the best disposal of surface water and groundwater control. The advantages and disadvantages of each of the common drain types are described below.

Ditch drains are typically the least costly means of providing drainage of surface water, i.e., storm runoff. If the stability of the ditch can be maintained, ditches may also be used to control groundwater levels. Unless lining is required, construction is accomplished with conventional earthwork equipment and no imported materials. Special track equipment can be used for excavating and maintaining lateral ditch drains and can provide for large flow capacity. Maintenance and modification of ditch drains after installation are simple because the drains are exposed at the surface. The disadvantage of using ditch drains is that they can only be constructed to a limited depth below the surrounding ground to avoid aggravating slope stability. Some of this geometry limitation can be alleviated by combining ditch drains with pipe culverts. If unlined, water flowing in the ditch will seep into the underlying subgrade. This is a particular problem for interceptor drains on slopes. Lining with pavement or membranes reduces the seepage but significantly adds to the cost of the drain. Because they are exposed, ditch drains typically require regular maintenance to clear vegetation and debris.

Pipe drains and French drains (referred to as trench drains) are similar in their uses and considerations. The pipe drain is a French drain with a pipe inside it to increase longitudinal flow capacity. The advantages of using these buried drains are that their designs are not as constrained by ground topography as is the surface ditch drain, and that buried drains can be routed beneath structures. French drains have a limited flow capacity, but this can be overcome by converting to a pipe drain configuration.

The principal difficulty in installing a pipe or French drain is providing adequate granular filter and collection layers around the pipe or stone. If the grain size characteristics of the soil to be dewatered vary, a variety of filter materials may be required. If fine-grained soils are encountered, multiple filter layers may be required to admit sufficient flow while preventing movement of soil particles. Even if filters can be properly designed, securing granular materials with the specified grain size at a reasonable cost may be difficult. Finally, placing a layered filter system into a trench is difficult to accomplish and control. The problems with graded filters have been generally eliminated by the adoption of filter fabric to line trench drains. Although the fabric relieves the requirements for graded filters, it adds about $1.50 to $2.00 per linear foot to the cost of the drain.

After trench drains are buried below ground, maintenance or repair of them is difficult. If a drainage pipe clogs, it sometimes can be cleared by flushing if adequate manhole accesses have been provided. If the collection zone of the drain becomes clogged with soil fines due to inadequate filters, the only repair technique available is excavation and reconstruction. If the drain underlies the track, reconstruction may be impractical.

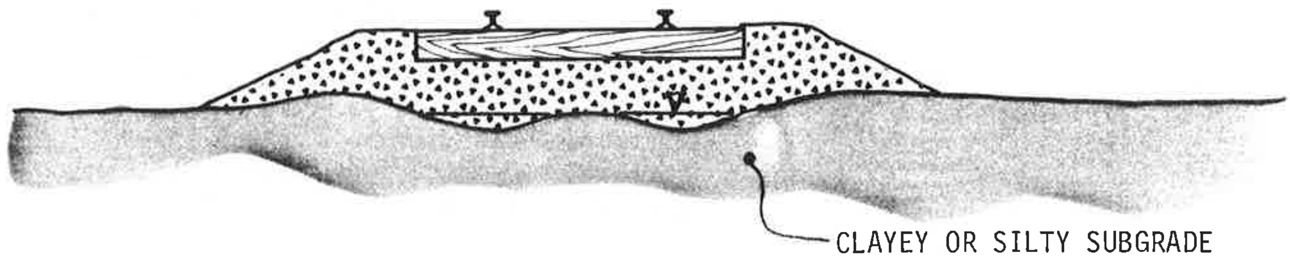
For any type of drainage structure, there are significant limitations in the design process. In remote locations, hydrologic data to determine design precipitation rates may not be available. Changes in the use of the ground adjacent to railroad track may alter the rate of storm runoff. One cause of such a change might be paving an area so that all rainfall runs off rapidly instead of some precipitation slowly seeping into natural ground.

The evaluation of subsurface flows is subject to even greater uncertainties due to geologic anomalies that often control subsurface flows and perhaps even surface drainage patterns and flows. Explorations may reveal some of these conditions, but not all. Both horizontal and vertical permeability generally must be evaluated to use a rational design procedure. Permeability is perhaps the most difficult soil property to determine with accuracy because permeability values may range over 10 orders of magnitude (clay to washed stone), and the horizontal permeability of a soil may be 100 or even 1000 times that of its vertical permeability. With this range in values and the knowledge that soil deposits are not uniform, it is recommended that an engineer experienced in the design of drainage systems should be involved in planning a drainage system.

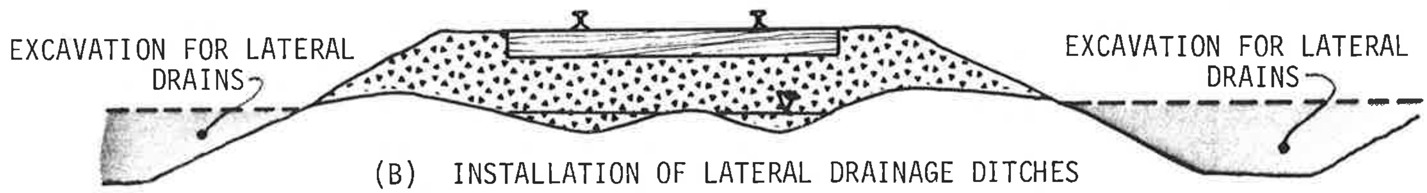
One of the factors affecting the performance of drainage systems is the condition of existing track subgrade. Figure 5-13a shows an idealized section through an existing small embankment in which ballast pockets and squeezes have developed. The greatest depth of the depressions is below the rails. Figure 5-13b shows what might happen if it were decided to add ditch drains only. The groundwater level below the track might be controlled by the ditch drains; however, surface infiltration would still be trapped in subgrade depressions. Recognizing that impermeable soils on the shoulders may inhibit drainage of infiltrating waters, it may be decided to plow the track shoulder and place new ballast. As shown in Figure 5-13c, such measures will improve runoff and drainage, but a depression in the subgrade remains because the deepest portion of squeezes or ballast pockets is below the rail, an area not affected by shoulder plowing. Excavating ditches and shoulder plowing would improve track performance in this case. However, much of the money may be wasted because the basic problem of ballast pockets and saturated subgrade immediately below the rails will still exist. If the above treatment is combined with ballast undercutting and installation of filter fabric, the ballast pocket progression might be completely halted.

5.2 IN-PLACE MODIFICATION

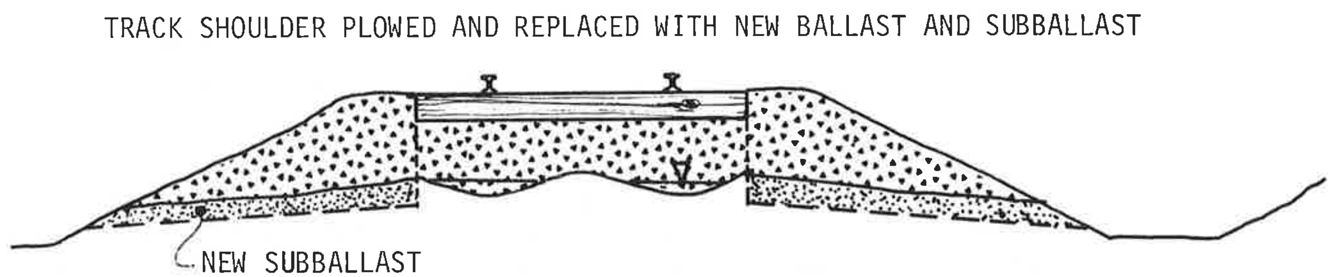
In-place modification generally can be accomplished with the track structure in-place and sometimes with only slow orders affecting train operations. Some of these methods, however, may cause gross distortion of the track, so that relining and tamping may be required before resuming train operation. In all cases, the methods treat the subgrade soil without removing it.



(A) BALLAST POCKETS OR SQUEEZES BELOW TRACK



(B) INSTALLATION OF LATERAL DRAINAGE DITCHES



NOTE: PLOWING AND REPLACING TRACK SHOULDER HAS SOME BENEFICIAL EFFECT ON DRAINAGE, BUT INFILTRATING RUNOFF CAN STILL BE TRAPPED IN EXISTING SUBGRADE DEPRESSIONS BELOW THE TRACK. DESPITE THE MAJOR DRAINAGE IMPROVEMENT PROGRAM, THE SUBGRADE MAY CONTINUE TO DETERIORATE AT ESSENTIALLY THE SAME RATE.

(C) PLOWING TRACK SHOULDER

FIGURE 5-13. LIMITATIONS OF DRAINAGE INSTALLATION

Grouting

Grouting of soil and rock deposits to alter engineering characteristics has been performed since the early 1800's, when slurried clay and hydraulic lime were pumped into subaqueous formations(1). Since that time, great improvements in grout types, injection procedures, equipment, control of work, and predictability of results have occurred. Grouting is performed to improve the strength characteristics of existing deposits, lower stratum permeability, or both. The number of possible applications, methods of injecting grout, and types of grout is vast; an increasing number of engineering problems are being solved using grouting techniques.

The two basic types of soil grouting are penetration grouting and compaction grouting. Penetration grouting occurs when the injected grout fills or penetrates the voids (pores) in the soil or rock mass. Compaction grouting is used to densify weak soil strata by displacing the soil with grout. The injected grout pushes the weak soil layer into a denser or more compact state. Figure 5-14 illustrates the basics of these grouting techniques. Railroad grouting was developed to treat specific subgrade problems by a combination of penetration and compaction grouting. Railroad grouting will be discussed as a separate technique in this section.

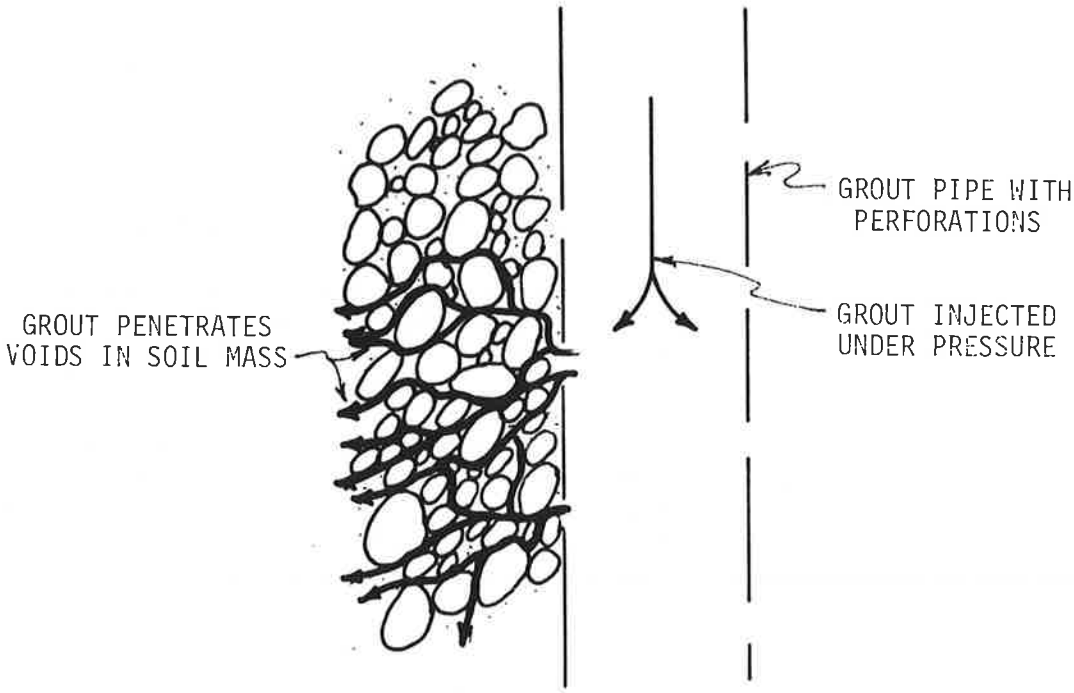
Penetration Grouting

This section briefly summarizes the principles of penetration grouting. More detailed information on penetration grouting is available from a variety of sources, including writings by Goldberg et al.(2), Ischy and Glossop(3), Einstein and Barvenik(4), and Sverdrup and Parcel(5).

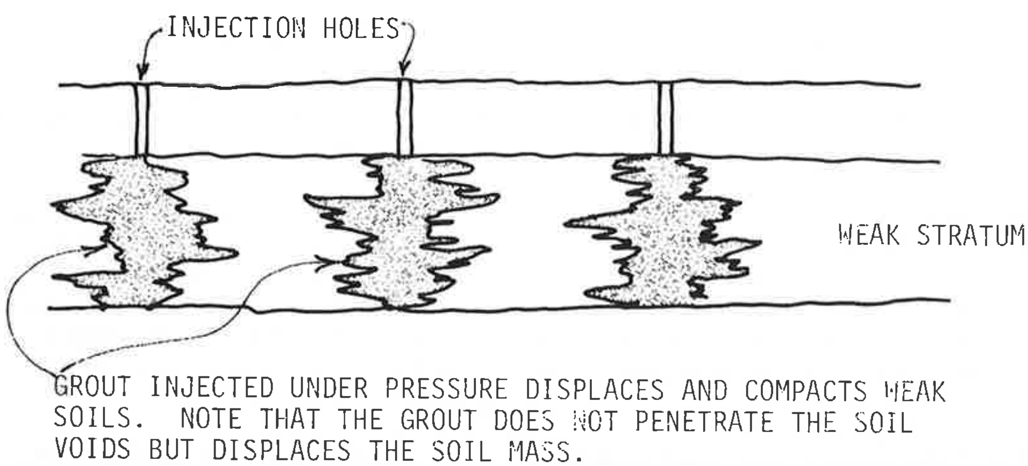
Penetration grouting involves the injection of grout into the voids of the soil skeleton. The intent of this type of grouting is to have the grout strengthen the soil mass or make the soil mass impermeable. Penetration grouting produces a grouted soil mass with relatively uniform strength and permeability characteristics. The extent to which this occurs depends upon how completely and uniformly the grout permeates the soil mass.

---

(1)E. Ischy and R. Glossop, "An Introduction to Alluvial Grouting," Proceedings of the Institution of Civil Engineers, Vol. 21, March 1962, pp. 449-473.
(2)D.T. Goldberg, W.E. Jaworski, and M.D. Gordon, "Lateral Support Systems and Underpinning," Volume III, Construction Methods, U.S. Dept. of Transportation, Federal Highway Administration, April 1976, FHWA-RD-75-130, pp. 335-365.
(3)E. Ischy and R. Glossop, "An Introduction to Alluvial Grouting."
(4)H.H. Einstein and M.J. Barvenik, "Grouting Applications in Civil Engineering," Vol. 1, Massachusetts Institute of Technology, Department of Civil Engineering, January 1975, pp. 192.
(5)Sverdrup and Parcel & Associates, "Cut-and-Cover Tunneling Techniques," U.S. Dept. of Transportation, Federal Highway Administration, Washington, D.C., 1973, FHWA-RD-73-40, pp. 107-119.



(A) PENETRATION GROUTING



(B) COMPACTION GROUTING

FIGURE 5-14. PRINCIPLES OF PENETRATION GROUTING AND COMPACTION GROUTING

Application - Penetration grouting is a technique that is limited to coarse grained soils -- sands and gravels. Particulate grouts, such as cement and clay, are effective penetrating grouts only in medium to coarse sands or gravel deposits without fines. Chemical grouts can penetrate finer-grained deposits than can particulate grouts, but no finer than a coarse silt (D_{10} > 0.01mm). Sodium silicate grout solutions are mainly applied in sands and fine gravels, while polymers or resins are used to grout silty sands and coarse silt. The low permeability of a clayey soil makes any kind of penetration grouting impractical generally. Figure 5-15 illustrates the general range of application for various grout types with respect to soil gradation.

In railroad work, grouting is most commonly used to stabilize a roadbed with ballast pockets or to stabilize embankment fills. An understanding of the principles of grouting will aid in evaluating the reasons why the procedures used by railroads have met with some success and will provide some of the limitations of these techniques.

Engineering - Explorations are needed to define the soil stratigraphy, particularly to assess the permeability of the soils to be grouted. Field and laboratory permeability tests, such as those described in Sections 2.3 and 5.1, are used to evaluate the soil permeability. Field permeability tests are generally considered more reliable than laboratory permeability tests. These tests may be expanded to include pressure injection of water into the soil. Gradation tests also give an indication of whether it is feasible to grout a soil. Soils containing less than 10 percent by weight passing the No. 200 sieve can generally be grouted. Penetration grouting of soils containing more than 10 percent passing the No. 200 sieve may be technically feasible, but it is often expensive to do so because of the long injection times required to achieve grout penetration.

Tables 5-2 and 5-3 and Figure 5-15 give basic ranges of soil permeability and grain size for which various grout types are applicable. Some grout types can be used only where a reduction of permeability is required and where no strength increase is needed. Clay or bentonite grouts are examples of grouts that are used to reduce permeability but give little strength increase. Normally, consolidation or strength grouting will also reduce soil permeability.

Particulate grouts are applicable only to coarse-grained soils. The ability of the grout to penetrate the soil is limited by the size of the particle in suspension and the size of the voids in the material to be grouted. Mitchell(1) defines a groutability ratio for soils as the ratio of the 15 percent size of the soil to the 85 percent size of the particulate grout.

---

(1)J. K. Mitchell, "In Place Treatment of Foundation Soils," Journal of the Soil Mechanics and Foundation Division, ASCE, Vol. 96, No. SM 1, 1970, p. 86.

TABLE 5-2. GENERAL RANGE OF GROUT APPLICABILITY

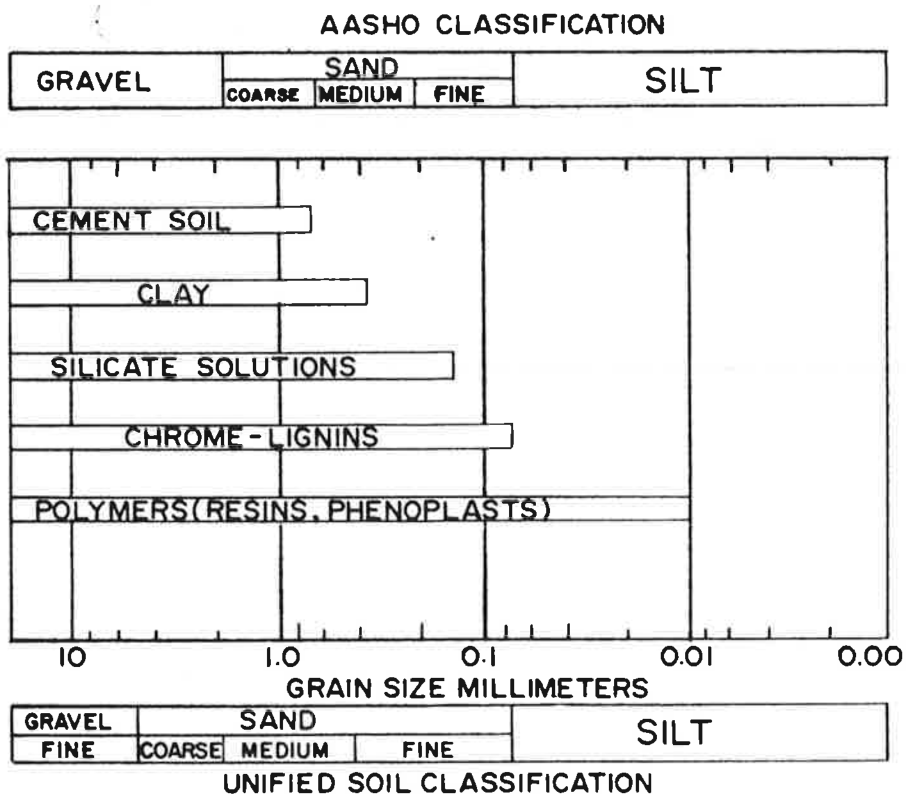
Type of Soil		Coarse Sands and Gravels	Medium to Fine Sands	Silty or Clayey Sands; Silts
Soil Properties	d_{10} Grain Size	>0.5mm	0.2mm to 0.5mm	<0.2mm
	Specific Surface Area	<100 cm^{-1}	100cm^{-1} to 1000cm^{-1}	>1000 cm^{-1}
	Permeability, k	>10^{-1} cm/s	10^{-3} cm/s to 10^{-1} cm/s	<10^{-5} cm/s
Series of Mix		Bingham Suspensions	Colloid (Gels)	Pure Solutions (Resins)
Consolidation Grouting		Portland Cement ($k > 10^{-2}$ cm/s) Aerated Mix	Double-shot Silica Gels Single-shot Silicates	Aminoplastic Phenoplastic
Impermeability Grouting		Aerated Mix Bentonite Gel Clay Gel Clay/Cement	Bentonite Gel Lignochromate Light Carongel Soft Silicagel Vulcanizable Oils Polyphenol	Acrylamide Aminoplastic Phenoplastic

Source: "Chemical Grouting For Paris Rapid Transit Tunnels," by J.J. Janin and G.F. LeSciellour. Year of first publication: 1970.

TABLE 5-3. PHYSICAL PROPERTIES OF CHEMICAL GROUTS

Class	Example	Viscosity cP	Gel Time Range Min.	Specific Gravity	Special Fields		
					Water-stopping	Consolidation	
					Fine Soil	Medium Strength	High Strentgh
Silica gel low concentration	Silicate-bicarbonate	1.5	0.1-300	1.02	X	X	
Silica gel high concentration	Silicate-formamide	4-40	5-300	1.10			X
Chrome lignin	TDM	2.5-4	5-120	1.10	X	X	
Vinyl polymer	AM-9	1.3	0.1-300	1.02	X	X	
Methylol bridge polymer	UF	6	5-300	1.08			X
Oil-based un-saturated fatty acid polymers	Polythixon FRD	10-80	25-360	0.99-1.05			X

Source: "Formulation and Selection of Chemical Grouts With Typical Examples of Field Use," by P.J. Neelands and A.N. James. Year of first publication: 1963.



Reproduced from Lateral Support Systems and Underpinning, Volume III. Construction, p. 348, by D.T. Goldberg et al. Year of first publication: 1976. (After"In-Place Treatment of Foundation Soils"by J.K. Mitchell, 1970.)

FIGURE 5-15. RANGES OF GROUT APPLICATION

For successful penetration grouting, the ratio should exceed 25. In practice, normal cement-based grouts are used only in coarse sands, while a pure bentonite grout might be injected into a medium sand.

Chemical grouts are used to grout fine to medium sands and, in some instances, coarse silt. Unlike particulate grouts that are injected as suspensions of solids, chemical grouts are solutions. Viscosity of grout, permeability of soil, gel time, and injection pressure control groutability. Properties of some chemical grouts are listed in Table 5-2.

The groutability of soil with chemical grout may be evaluated using the equation presented by Ischy and Glossop in 1962:

$$t = \frac{\alpha n}{3khr_0}(R^3 - r_0^3)$$

where

R = radius of grout distribution (sphere)
r_0 = radius of injection pipe
n = porosity of soil
k = soil permeability
α = ratio of grout viscosity to that of water
h = piezometric head in the grout pipe
t = time of grouting

By inserting typical values into this equation, the problem encountered with injecting low permeability soils is apparent. Consider a pipe radius of one inch, a soil permeability of 10^{-5} cm/sec (silt or clay), a pressure head of 10 psi, and a soil porosity of 0.4. Water would penetrate approximatey 6.2 inches out from the pipe in a grouting time of 30 minutes. Clearly, this does not represent a significant amount of grout penetration. Clayey soils may have permeabilities two orders of magnitude lower than that used in this example.

Description - Particulate grout slurries are made up of variable mixtures of cement, clay, sand, fly ash, and water. The ratios of these materials depend upon the stratum to be grouted and the properties desired after grouting. Injection is achieved through an open hole or through a perforated pipe. Injection pressures are controlled to minimize heaving at the ground surface, yet must provide adequate penetration. Grouting generally continues until refusal occurs.

Chemical grouts can be injected in either a one-shot or two-shot process. Early chemical grouting used two-shot injections, but improvements in grouting technology have resulted in the increased use of one-shot injections. Two-shot injections involve an initial injection with a chemical followed by a second injection with an activating chemical. Sodium silicate and calcium chloride are the most common chemicals used, but others are available. The disadvantages of the two-shot method are: (1) that it requires two injections, which is more time-consuming, and (2) the reaction between the chemicals is rapid, which hinders full grout penetration. The principal advantage is that higher strength grouts are possible than with a single injection.

The typical procedure for injecting penetration grouts is to drive a perforated or unperforated pipe to the desired depth. When perforated pipe is used, normally only the lower end of the pipe has perforations. Unperforated pipe is driven with a point which is detached after driving; injection occurs in about one-foot intervals as the pipe is withdrawn. Grout pressures are usually limited to 1 psi per foot of depth below the ground surface to prevent grout breakout and fracturing of the ground surface.

Cost - Penetration grouting is expensive. Particulate grouts are less expensive than chemical grouts because the materials are cheaper, and the installation equipment and methods are less complex. The cost of cement grout material is approximately \$0.50 to \$1.30 per cubic foot injected, while the cost of chemical grout material varies from \$1.50 to \$7.00 per cubic foot(1). Installation costs are quite variable, depending upon time of grouting, type of equipment, depth, and many other factors. Total grouting costs for cement grouts are on the order of \$13.50 to \$35.00 per cubic yard of grouted soil, while costs for chemical grouting are approximately \$40 to \$190 per cubic yard of grouted soil.

Compaction Grouting

Compaction grouting is applicable where loose or weak soil strata, such as loose fills, loose natural granular soils, or cohesive soils, are present within approximately 20 feet of the ground surface. Compaction grouting is used when the weak strata cannot be effectively strengthened through the use of penetration grouting. This technique is applicable to silty sands, silts, and some clays. Because saturated cohesive deposits build up pore pressure in response to loading, the technique has met with limited success when used in these soil types.

Since the method involves the injection of a very stiff cement grout under pressures normally ranging from 50 to 500 psi, there are instances where it cannot be used. Generally, it has been found ineffective within 4 to 6 feet of the ground surface. Surface breakout or fracturing occurs before effective compaction can be achieved. Similarly, near the edge of slopes there is insufficient lateral resistance to confine the grout. Generally, injection holes must be 10 to 50 feet from the edge of a slope to achieve effective compaction.

---

(1)D.T. Goldberg, W.E. Jaworski, and M.D. Gordon, "Lateral Support Systems and Underpinning," Vol. 1 - Design and Construction, U.S. Dept. of Transportation, Federal Highway Administration, Washington, D.C., April 1976, FHWA-RD-75-128, p. 109.

One of the more common applications of compaction grouting has been to raise settled structures. Experienced contractors can raise residential or light commercial structure grades as much as 0.3m (1 foot). In this respect, the technique is very similar to mud jacking, yet it differs from mud jacking in that both the soil and the structure are lifted, rather than just the structure.

Description - Compaction grouting is used to densify weak strata by applying force to the strata. The force is applied by grout emanating radially from an injection pipe. Unlike penetration grouting, there is a distinct interface between the grout mass and the soil mass.

Compaction grouting involves the driving of perforated or unperforated pipe to the required depth. Usually this depth is just into firm materials underlying the weak layer. When open-ended pipe is used, the bottom plug is driven out, and the grout is injected as the injection pipe is withdrawn. The pipe is withdrawn a few feet at a time until the weak layer is grouted. Grouting is continued to within 4 to 6 feet of the ground surface. Grouting is stopped at each point when there is little or no additional penetration of grout or when movement of the ground surface, grout breakout, or fracturing of the ground is noted. A technically more desirable method is to inject grout from the top down, using casings cemented into the injection hole. After each injection of 5 to 8 feet, the injection pipe is advanced through the cement grout for the next injection. This method reduces the possibility of premature breakout along the pipe.

The injection pipes are spaced on a grid, ranging from 1.5m to 5m (5 to 15 feet), with the final spacing being determined by field observations. Initially, a 5m to 6m (15-foot to 20-foot) spacing pattern will be grouted. Intermediate holes will then be grouted. If these intermediate holes accept very little or no grout, the spacing between holes will remain the same with no intermediate injections. If intermediate holes accept significant grout quantities, grouting at this reduced spacing is performed. The spacing of injection holes is adjusted until the optimum spacing is obtained. The injection pattern is controlled by actual field observations, and careful recording of grout takes is essential to effective application.

Compaction grout is a very stiff mixture of cement, sand, and occasional additives, including fly ash, clay, or asphalt emulsions. The asphalt emulsions give the grout more fluidity to help it to flow through the hoses and pipes. The equipment needed to employ the method consists of a grout mixer, an injection pump, and a drill rig to advance the hole and injection pipe. This equipment is commonly available, but the compaction grouting technique requires experienced operators familiar with the procedures.

Engineering - Compaction grouting is a technique that relies largely on field observations. The source of the problem must be identified, but detailed explorations are not required normally, since the grout injection holes serve as additional explorations. The main engineering effort occurs during the grouting, when grout takes must be carefully recorded and related to spacing and soil conditions to obtain the optimum grout hole spacing that will yield satisfactory results.

Railroad Grouting

Application - Railroad grouting has been confined to solving two basic subgrade instability problems -- ballast pockets and unstable fills. As reported by Smith and Peck(1), railroad grouting techniques have not been used very successfully to stabilize natural soil deposits.

Section 2.7 noted that the vast majority of subgrade problems occur in saturated cohesive or fine-grained granular deposits. The previous sections on penetration and compaction grouting indicate that neither method is applicable to saturated cohesive soils because: (1) these soils are not sufficiently permeable to allow grout to penetrate the soil skeleton, and (2) compaction grouting causes a buildup of pore pressure that prevents densification.

Description - Railroad grouting is performed using a mixture of sand, cement, and water that is pumped into the ground under pressure. The grout is installed through a pipe under pressure using either pneumatic or hydraulic equipment. Open end or perforated pipe can be used, although an open end pipe with a knockout plug is more common.

Pneumatic injection equipment consists of a pressurized mixing tank with a central shaft and mixing paddles. The equipment was first developed by railroads but is now commercially produced. Open end pipes are normally used for injection, with water pumped through the pipe before injecting grout. Hydraulic equipment similar to mud jacking equipment can also be used. The hydraulic equipment has the advantage of offering greater production rates, but the equipment is more expensive. Pneumatic pressure equipment normally is used for ballast pocket (shallow) grouting, while hydraulic equipment is applied more often to deep fill stabilization.

Ballast Pocket Grouting - The general procedure for grouting ballast pockets is to drive pipe through the ballast and slightly into the subgrade. Water is pumped through the pipe after the drive plug is knocked out. The water opens up passages for the grout to flow, particularly in sand or cinder subballasts. The grout is then pumped at pressures of 40 to 60 psi until

---

(1)R. Smith and R.B. Peck, "Stabilization by Pressure Grouting on American Railroads," Geotechnique, Vol. V, No. 3, 1955, p. 243.

breakout or track lifting occurs. Large grout takes are unusual when grouting for ballast pockets, making pneumatic equipment practical. Points driven too far into clay subgrades will not accept much grout and must be withdrawn a few inches and regrouted.

Injection holes are spaced 1.5m to 3m (5-feet to 10-feet) apart at alternating patterns on either side of the track. The points should be driven to the base of the pocket either from between ties or at angles from the ends of ties. When performed from the side, injection can be done without disruption to train traffic. Figure 5-16 illustrates a typical layout for grouting ballast pockets.

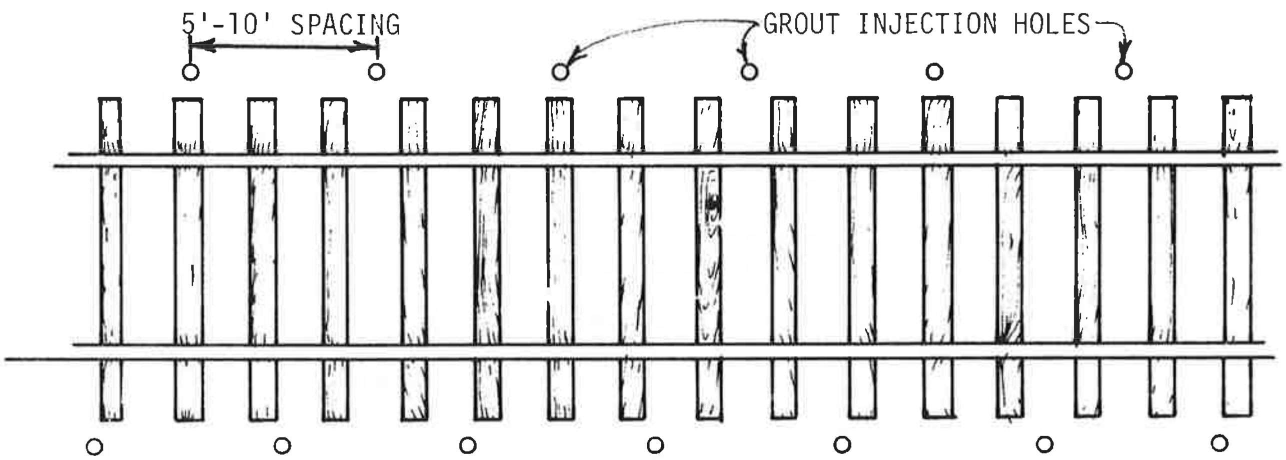
Two beneficial situations may arise as a result of ballast pocket grouting by common railroad grouting techniques. First, the grouted ballast may improve track performance by stiffening the ballast and reducing water access to the subgrade. Second, a thin layer of grout may form between the subgrade and the ballast or subballast. Since the grouts used are too coarse to penetrate strata other than crushed stone, the grout will seek out a plane of weakness in the substructure and flow along this path. It may form a thin grouted zone between the subgrade and subballast. This zone, if developed, might serve to spread out loads from the ballast to the subgrade to reduce the amount of moisture getting to the subgrade and thereby limit further deterioration of the subgrade. How effectively the grout keeps water from the ballast pocket subgrade depends upon the final grout zone shape.

Embankment Grouting - The procedures used to grout unstable fills are similar to those for ballast pockets, except that the grouting typically extends to greater depths, and the injection spacing differs. It has been observed that railroad embankment instability sometimes occurs because of defects in the fill, rather than defects or weaknesses in the underlying natural soils. Often, voids or seams are present in the fill if it was placed by dumping without compaction. These voids fill with water, adding weight, softening the soils, and increasing pore pressures in the fill. Because of these voids, the grout takes can often be quite large. Grouting in holes accepting more than 5 cubic meters (150 cubic feet) of grout is normally stopped until adjacent holes are grouted. Regrouting of this hole would then be performed.

Grouting normally starts at the bottom of the slope and works upward. The weak or unstable plane must be intersected by the grout pipes; the grout pipes are normally driven to natural soils, at least at the toe of the slope. Injection hole spacing is normally at staggered 10-foot intervals, both horizontally and vertically. For high embankments, several injections up the side of an embankment may be required.

Figure 5-17 illustrates two cases where grouting might be used successfully in fill stabilization. In the first case, the grout fills open voids in the fill while, in the process, expelling water and providing additional strength. In the second case, the grout fills seams of weakness within the fill. Once again, water is expelled as the grout is injected. After the

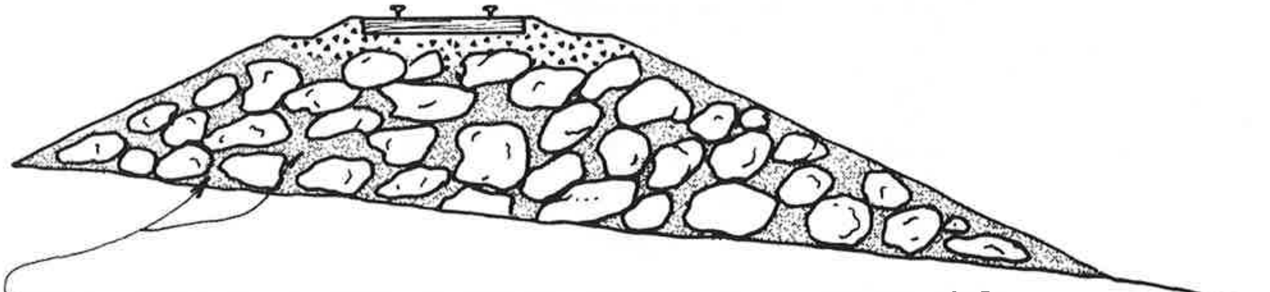
(A) VERTICAL SECTION OF INJECTION PIPE INSTALLATION PATTERN



(B) PLAN OF GROUT HOLE SPACING FOR BALLAST POCKET GROUTING

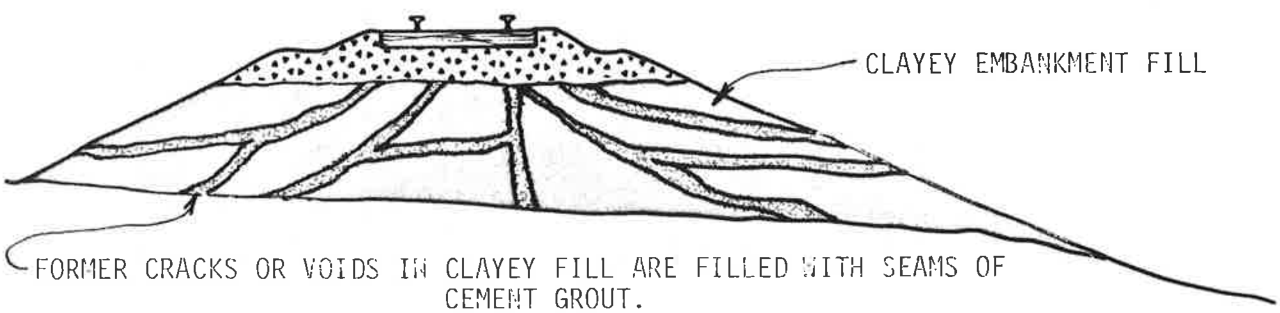
FIGURE 5-16. INJECTION PATTERN FOR BALLAST POCKET GROUTING

EMBANKMENT CONSTRUCTED OF LUMPY CLAY OR BOULDER FILL WITH LARGE OPEN VOIDS



CEMENT GROUT FILLS VOIDS IN BOULDER OR CLAY FILL. EXCESS WATER REMOVED AND FILL CEMENTED TOGETHER.

(A) FILLING OPEN VOIDS IN FILLS.



FORMER CRACKS OR VOIDS IN CLAYEY FILL ARE FILLED WITH SEAMS OF CEMENT GROUT.

EXCESS WATER IS EXPELLED FROM EMBANKMENT AND PATHWAYS FOR FUTURE WATER PENETRATION ARE BLOCKED BY CEMENT GROUT. THE NET RESULT IS TO REDUCE THE AMOUNT OF WATER ENTERING THE FILL SUBGRADE.

(B) FILLING SEAMS IN EMBANKMENT FILL

FIGURE 5-17. RESULTS OF CEMENT GROUTING TO IMPROVE EMBANKMENT STABILITY grout hardens, it forms a barrier to further water penetration into the fill. Expelling excess water and stabilizing the moisture content of the fill (particularly along planes of weakness) are probably the most beneficial effects of grouting. Although there are no conclusive data to support this assumption, it is also possible that grouting may compact the fill soils, raising density and strength.

Cost - As with other grouting methods, the cost of railroad grouting is high. In shallow applications (injection depth less than 6m (20 feet) the cost is approximately $10 to $15 per track foot. Grouting embankments 100 feet high might cost as much as $100 to $125 per track foot. Studies by several railroads have indicated that the costs of grout injection have been recovered in maintenance savings within three years. This does not include savings resulting from raising speeds. The benefits have been observed to continue well beyond the first three-year period.

Discussion - Grouting as commonly used in railroad work is a hybrid of the two basic grouting techniques, penetration and compaction grouting. While the installation techniques are more closely related to compaction grouting, the actual results are often penetration of ballast and displacement of void water.

The principal advantage of grouting in railroad work is that it is a nondisruptive method. Although expensive, it appears to be cost effective. There is no good theoretical basis to explain why the technique works. It is difficult to predict how successful the program will be before it is undertaken. Track maintentance after grouting is often required to remove cemented ballast from the zone immediately below the track and ties. Hard spots may also form below track. If the ballast becomes cemented, further maintenance or rehabilitation is much more difficult.

The fact that the techniques do work makes them a viable method of stabilizing railroad subgrade. More research into why railroad grouting is effective seems warranted, and further studies as to the viability of using "conventional" grouting techniques should be performed.

Lime Slurry Pressure Injection

Application - Lime stabilization is applicable to fine-grained, plastic soils. While virtually all fine-grained soils react in some manner with lime, granular soils without clay are not affected by lime treatment(1). Lime is added to cohesive soils to reduce their plasticity and to reduce their swell potential. A secondary objective in lime treatments is to increase

---

(1)M.R. Thompson and Q.L. Robnett, "Pressure-Injected Lime for Treatment of Swelling Soils," Transportation Research Record, No. 568, 1976, p. 25.

soil strength. In railroad track applications, increase in soil strength is often the primary objective in the treatment of squeezes and ballast pockets.

Description - Lime slurry pressure injection (LSPI) is a method that has been developed in recent years to stabilize clayey soils. The technique has been used to stabilize soils below building foundations, highway pavements, and railroad tracks. To date, LSPI has met with mixed success on these projects. In several instances, the users have reported successful results; while in other cases, no apparent improvement in subgrade performance was noted.

Lime has long been recognized as an effective additive for stabilizing clayey soils. It is most commonly used during placement as compacted fill. The lime is thoroughly mixed with the soils before compaction to reduce the plasticity and swell potential of plastic clays, as discussed in Section 5.4.

The three basic mechanisms by which lime alters the properties of a cohesive soil are through cation exchange, agglomeration-flocculation, and a pozzolanic or cementing reaction. Cation exchange involves the replacement of sodium and potassium ions in the clay with calcium ions from the lime. Agglomeration and flocculation yield an apparent change in soil texture, with the clay particles clumping together. The pozzolanic reaction is a reaction between silica or aluminum and lime to form various cementing agents. Cation exchange and agglomeration-flocculation occur in almost all lime-treated cohesive soils and result in a reduction in plasticity and a decrease in swelling and shrinking characteristics. The pozzolanic cementing reaction does not occur in all lime-treated soils. Soils experiencing a pozzolanic reaction with lime are generally referred to as lime-reactive. A significant strength increase is achieved in lime-reactive soils, and lime treatment for stabilization is most effective in these soils. One factor influencing the effectiveness of any lime treatment is the temperature at which the reaction occurs. At temperatures below 5°C (40°F), lime-soil reactions are negligible, while best results are obtained at temperatures above 15°C (60°F).

Removing existing soils, mixing them with lime, and replacing the soil-lime mixture is an expensive soil stabilization procedure, particularly if several feet of the subgrade soil must be stabilized. Clearly, for in-service track application, excavation and replacement is a completely disruptive and expensive subgrade stabilization procedure. To improve the economics of lime stabilization, several attempts have been made to stabilize clay subgrades without excavating and mechanically mixing soils and lime.

Drill hole lime injection was the forerunner of the LSPI method. This technique involves the excavation of 150mm to 300mm (6-inch to 12-inch) diameter holes into natural subgrade soils. These holes are then filled with hydrated lime and water and backfilled with the excavated soils. The intent of this method is to have the lime permeate or diffuse into the soil mass to stabilize it. It was observed that little penetration of the lime into the subgrade occurred using this technique; however, some success was noted despite the lack of diffusion(1). LSPI was developed to improve the penetration of lime into the soil mass. This section discusses the LSPI method, although some parts are applicable to the drill-hole method.

The basic LSPI method consists of injecting a lime slurry under pressure into a previously identified, troublesome subgrade layer. These remarks briefly summarize the method most commonly used to inject lime slurry under pressure into railroad subgrades. Blacklock and Lawson (1977) as revised by Ledbetter (1978) describe the installation procedures in greater detail in the Handbook for Railroad Track Stabilization Using Lime Slurry Pressure Injection(2).

Although either quicklime or hydrated lime can be used as a source of lime, hydrated lime is more stable and safer to use. Hydrated lime is readily available from a number of construction suppliers. Typical construction-grade hydrated lime is normally adequate for LSPI treatments. The Handbook presents detailed specification criteria for hydrated lime.

Before injection, the hydrated lime must be mixed with water to form the slurry. The slurry can be mixed in a large tank with paddle wheel agitators or in a smaller blending truck. The large tanks have the advantage of being able to mix larger quantities of slurry, with better control of slurry consistency. A slurry mix of 2.5 to 3 pounds of lime per gallon of water is most commonly used, although mixes as thin as 2 pounds of lime per gallon of water have been used in dry soils.

For in-service track application, the typical injection equipment consists of a truck with hyrail wheels, a 1,500- to 2,000-gallon tank, three injection rods, and appropriate pump equipment to inject the slurry through the three injection rods located at the rear of the truck. One rod is located over the track centerline, with the other two rods located about five feet on either side of the center rod, just beyond the ends of the ties. The rods are pushed hydraulically into the subgrade, and the slurry is injected at pressures between 50 and 250 psi, with pressures of about 150 psi being most commonly used. Injection normally proceeds from the top downward at intervals of 12 inches to 18 inches. The actual vertical injection spacing can be varied to suit soil conditions and desired slurry takes. Injection continues until refusal. Refusal is normally defined by slurry emanating at the track surface, in ditches, or through adjacent slurry injection holes.

Spacing of injection points along the track is variable and depends largely on track and subgrade conditions. Typically, injections are made between every second or third tie. Ultimately, the spacing is controlled

(1) J.B. Farris, "Drill-Lime Treatment of Shallow Railway Subgrade Failures in Expansive Clays," American Railway Engineering Bulletin, No. 26, Vol. 71, February 1970, pp. 574-579.
(2) Blacklock, J.R. and C.H. Lawson (original authors) as revised by R.H. Ledbetter, Handbook for Railroad Track Stabilization Using Lime Slurry Pressure Injection, U.S. Dept. of Transportation, Federal Railroad Administration, Washington, D.C., Revised 1979, FRA/ORD-77/30, p. 92.

by the amount of slurry injected into the soil mass that is to be stabilized. If the original spacing of every second or third tie does not result in enough slurry take, intermediate points are injected.

Engineering - The ideal lime treatment is a complete and uniform mixture of lime and soil. A uniform soil-lime mixture cannot be achieved during the LSPI method because the lime cannot penetrate the soil structure. The voids in intact fine- grained soils are too small to allow individual lime particles to penetrate the soil mass. As was discussed in the previous section on grouting, even a fluid grout cannot penetrate the voids of a fine-grained soil within the normal grouting time of a few minutes.

Injecting lime slurry under pressure results in slurry penetration along fissures, cracks, root holes, sand seams, bedding planes, and other discontinuities in the soil. In addition, the injection pressures may open new paths along planes of weakness in the soil. The result of LSPI is to form a network of lime seams throughout the soil mass. This does not constitute a uniform lime-soil mix, although some diffusion of lime from the seams into the soil mass does occur with time(1).

Explorations are a critical aspect of any subgrade stabilization scheme. The source and extent of the problem must be determined through surface and subsurface investigations. Blacklock, Lawson and Ledbetter give recommendations concerning the spacing and depth of borings for subgrade stabilization problems where LSPI might be used. The following table presents their basic recommendations for number of borings for different lengths of problem zones. Boring programs should be planned and supervised by an experienced engineer who can adjust the program in progress to meet the needs of the project.

NUMBER OF BORINGS

Length of Problem Track	Number of Borings
0 to 1,000 feet	2 + length/250'
1,000 to 4,000 feet	6 + (length-1,000')/300'
4,000 to 10,000 feet	16 + (length-4,000')/400'

An important part of any exploration program where LSPI is being considered is obtaining undisturbed samples of the subgrade soils. Since pre-existing flow channels in the soil are critical to lime slurry penetration, the soil should be observed carefully for fractures, cracks, sand seams, etc. that will promote the flow of lime slurry. These undisturbed samples are also

---

(1) H.L. Lundy and B.J. Greenfield, "Evaluation of Deep In-Situ Soil Stabilization by High-Pressure Lime-Slurry Injection," Highway Research Record, No. 235, 1968, pp. 27-35.

used to assess the effect that lime slurry has on the soil properties. Cracks, fissures, and other voids in the soil may be observed more easily in test pits than in a smaller undisturbed sample, although interference with train operations may restrict execution of large-scale explorations close to the track.

Laboratory tests used to evaluate the potential effectiveness of lime stabilization include Atterberg Limits, swell-shrink tests, consolidation tests, and triaxial strength testing. Testing should be performed in both control (unaltered) and lime-treated samples. Lime can be added to samples by mechanical mixing or by inoculating undisturbed samples with hypodermic needles. Ledbetter (1979) describes the laboratory testing procedures that have been used to evaluate the potential effectiveness of LSPI treatment. The pH test described in Section 5.4 for lime admixture stabilization may add information on lime reactivity of soil.

The two basic considerations in deciding whether to use LSPI are: (1) the presence of flow paths through which the lime slurry can form a network of lime seams, and (2) the changes in soil properties that occur because of lime treatment. Laboratory testing indicates what the changes in soil properties will be, but the tests are far from confident indicators of treatment success. The presence of flow paths in the subgrade is more important to LSPI success than is the amount of change in soil properties. If the lime cannot be introduced into the subgrade, its degree of reactivity with the soil is immaterial. Studies(1) have indicated that the major benefits of lime treatment are the stability of the soil's moisture content and the formation of a moisture barrier. This moisture barrier limits the amount of excess water reaching the soil mass via the discontinuities, thereby limiting swell and strength degradation. Prewetting of the soil during slurry injection also apparently aids stabilization of subgrade soils susceptible to swell.

Cost - LSPI treatment costs vary greatly with the amount of lime injected per track foot. J. B. Farris (2) of the Southern Railroad estimated that LSPI treatments normally cost $5.00 to $6.00 per track foot. Ahlf reported a 1980 cost of $6.62 per track foot (3).

Discussion - The principal advantages of the LSPI method are: (1) it is a relatively inexpensive method for treating poor clayey subgrades, and (2) track disruption is minimal -- the equipment operates on the track, but the track structure is not removed. LSPI appears useful in reducing the

---

(1) M.R. Thompson and Q.L. Robnett, "Pressure-Injected Lime for Treatment of Swelling Soils," Transportation Research Record, No. 568, p. 32, 1976.
(2) J.B. Farris, Personal Communication, 1979.
(3) R.E. Ahlf, "Matching M/W Practice to Required Use of Track -- Part IV: The Cost," Railway Track and Structures, Vol. 75, No. 11, January 1980, p. 22.

swell potential of plastic soils and may improve the strength characteristics of some soils. In some cases, LSPI treatments have been judged effective stabilization measures.

The two major limitations of the LSPI method are: (1) the reasons why it works are not well understood, and (2) it is difficult to predict its effect without a full-scale trial. Laboratory tests and field observations give the engineer some general guidelines for estimating the potential effectiveness of the method. Even where successful application of the method has been reported, there is some doubt about what improvement was generated by LSPI. In many of these cases, other remedial work was performed at approximately the same time. Therefore, it is not known to what extent LSPI treatments improved performance and to what extent the other remedial measures did. An observed effect is that the lime may react with the fines in a fouled ballast, leading to partially cemented ballast. Creating a stiffer ballast may account for some of the performance improvement that has been observed.

There are also some environmental concerns with LSPI. Plant and fish life may be adversely affected if spills occur or if much lime seeps out of the treated subgrade. Contamination of drinking supplies is also possible. Care must be employed in the handling and use of these materials.

In conclusion, the LSPI method apparently has been effective in improving subgrade performance in some cases; however, the magnitude of the effect is questionable because of other remedial measures performed at the same time. More research and study of the method is required to assess the mechanisms by which lime slurry does improve subgrade performance. The limitations of the laboratory tests suggest that only full-scale field tests of lime slurry injection can confidently predict LSPI effectiveness. Future research must address the actual performance of lime-treated subgrades.

Deep Densification

Application - Deep densification--which refers to using vibratory or impact forces to densify soil in place--is mainly used with loose granular soils. Some unsaturated cohesive soils can also be treated by deep densification. The upper limit on the amount of fines (i.e., particles passing a No. 200 sieve) is about 20 percent for most deep densification methods. Particle size gradation suitable for densification by vibroflotation is shown in Figure 5-18. Deep densification is used to limit settlement of soil subject to vibratory loading from trains or earthquakes.

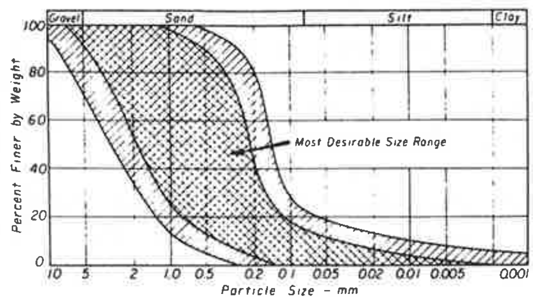


FIGURE 5-18. RANGE OF SOIL GRAIN SIZES SUITABLE FOR COMPACTION BY VIBROFLOTATION

Description - Deep compaction can be accomplished by several means including vibroflotation, vibratory compaction piles, dynamic compaction, and blasting. Each of these methods is described below.

Vibroflotation involves using a Vibroflot®, a cylindrical penetrator about 0.4m (15 inches) in diameter and about 2m (6 feet) long. An eccentric weight inside the cylinder develops a horizontal centrifugal force of about 9 tonnes (10 tons) at 30 hertz. The total weight of the vibroflot is about 1.8 tonnes (2 tons). Dynamic displacement is about 20mm (3/4 inch). In addition to the vibratory action, the Vibroflot is equipped with water jets that ensure saturation of the surrounding soil and add seepage pressure of the water to the vibration to densify the soil. Complete saturation by the water jets removes any apparent soil cohesion, which aids in rearrangement of the particles.

The Vibroflot is attached to a pipe and is suspended by cable in a pile driving lead. The maximum depth of soil treated typically is 8m (25 feet) with a maximum reported treatment depth of 19m (62 feet). After the Vibroflot reaches maximum treatment depth, the hole is backfilled in stages with clean, coarse, granular soil that is compacted with the Vibroflot. The resulting column of dense soil is about 1m (3 feet) in diameter, but the soil outside the densified column also is densified to a radius of 1.5m to 3m (5 feet to 10 feet). In preparing foundations for buildings, treatment centers typically are spaced 2m to 4m (6 feet to 13 feet) apart, depending on the degree of densification required and the properties of the soil, particularly the fines content. Spacings closer to 4m are judged appropriate for densifying sands beneath railroad track. A summary of the vibroflotation process was provided in 1970 by J. K. Mitchell in "In-Place Treatment of Foundation Soils", and in 1975 by D. A. Greenwood in "Vibroflotation: Rationale for Design and Practice."

Vibratory compaction pile -- sometimes known as Terra Probe® -- comprises a thick walled steel pipe with open ends, about 0.8m (30 inches) in diameter and long enough to reach the soils to be densified. The pipe is attached to a vibratory pile-driving hammer to supply energy. As described in 1974 by R. D. Anderson in "New Method for Deep Sand Vibratory Compaction," by vibrating the open pipe into the ground and withdrawing it, this procedure densifies the soil within the pipe probe and a short distance outside the pipe. Probes are spaced approximately the same distance as for Vibroflotation.

Dynamic compaction involves densifying soil in place by repeatedly dropping heavy weights on the surface using a crane. The weights typically used range from 10 tons to 20 tons (9.1 to 18.2 tonnes). The drop height typically is 15 feet to 50 feet (5m to 15m). Compactions are spaced 3 feet to 10 feet (1m to 3m) apart and multiple drops are used. The depth of effective treatment depends on the energy input by the weights. In 1980, G. A. Leonards et al. suggested in "Dynamic Compaction of Granular Soils," that the effective depth of treatment may be calculated by the relation

$$D = \frac{1}{2} \sqrt{WH}$$

where

D = depth of influence in meters
W = falling weight in tonnes
H = height of drop in meters.

The effective depth of dynamic compaction is influenced by details of the soil profile. If the loose soil is underlain by a stiff layer, the compaction energy is confined and a greater depth can be treated. If a soft clay underlies the loose surface layer, the clay will absorb much of the energy making it difficult to densify soil close to the clay layer. The maximum depth that can be treated is about 10m (30 feet); 6m (20 feet) may be the maximum practical depth.

Deep compaction does not require the soil to be saturated. Because compactions are applied more slowly than by vibratory methods, soils with a higher fines content -- silts and clays -- can be densified effectively. The method has been used to treat cinder, ash, and rubble fill to prepare for shallow foundations to support a six-story structure.

Blasting can be used to densify loose, saturated, fine sands; although some success in silty sands has also been reported, as summarized by J. K. Mitchell in "In-Place Treatment of Foundation Soils." Blasts typically are spaced 10 feet to 25 feet apart. Repeated shots are more effective than a single, large blast; however, the spacing of successive shots should be progressively subdivided. The method is particularly well suited to treating deep deposits.

After all deep compaction treatments, the surface soils require densification by surface rolling with a heavy vibratory compactor. A mat of controlled, compacted granular fill several feet thick should be placed at the surface after in-place densification to bridge irregularities.

Engineering - Specific engineering factors associated with deep densification include establishment of groundwater levels and measurement of soil gradation characteristics by sieve analysis. Design of a deep densification program is not based on rational analysis but on rules developed by experience. Each specialty contractor has his own proprietary design method. The principal engineering factor is monitoring the densification achieved in the field. Densification should be measured by "before and after" tests. The standard penetration test, the cone penetration test, and the pressuremeter test have all been used to measure in-situ density changes of soils subjected to deep densification. For major densification projects and at the early stages of all projects, a test program with varied probe spacing and other densification variables should be carried out to determine that the required densification is being accomplished without excessive compaction effort. For major programs, design-phase field tests should be carried out to provide field comparison of competing methods.

Cost - The cost of densification is a function of soil characteristics, depth to be treated, and other factors. If imported material must be provided, such as is sometimes required for vibroflotation, this cost must be added. Because all methods result in settlement of the area, regrading with compacted fill typically is required. As a general rule of thumb, deep densification costs about $0.50 to $2.50 per cubic yard of material treated.

Discussion - The principal advantage of using deep densification methods is that a thick deposit of granular soil can be densified in place. The cost is substantially less than for excavation and recompaction. The disadvantage is that the effects of deep densification vary and must be confirmed by "before and after" in-situ density tests. Clean, granular soils and unsaturated cohesive soils are the best candidates for deep densification. Granular soils, even though loose, do not cause unsatisfactory track performance frequently except for causing fouled ballast which is treated best by some of the methods discussed in Section 5.3.

It may be possible to accomplish deep densification with some success with the track structure in place, provided the equipment can operate on both sides of each track. However, the densification will be accompanied by gross settlement of the track which will require resurfacing and alignment prior to placing the line back in service. Deep densification probably is only practicable for new construction or when completely rebuilding the track.

Preloading

Application - Preloading is a method that can be used to treat saturated cohesive soils that only respond to load increases applied over long periods of time. Preloading is used to accelerate settlement so that the rate of displacements during operation of the track will be small. Normally consolidated soils, as discussed in Section 2.2, will require preload treatment more often than overconsolidated soils.

Description - A typical compression-time relation for magnitude of consolidation settlement is shown in Figure 5-19. The quantitative scales of this plot are not important. Associated with average settlement is some variation of settlement from place to place, this differential settlement will be some fraction of the average settlement magnitude, typically one-third to three--quarters. It is this differential settlement that leads to track geometry degradation. Preloading accomplishes the early stages of consolidation, such as that occurring from 0 to 3T, before the track structure is in place. The rate of settlement after 3T is slow enough that track geometry will not be affected by additional settlement.

The easiest way to accomplish preloading is simply by constructing embankments and then waiting a period of time before final grading and construction of the track structure. The waiting time can be evaluated by procedures discussed in Section 2.2. Sometimes the time required to reduce future settlement rates to an acceptable range is greater than can be practicably accepted. It may be as great as several years or tens of years. To accelerate consolidation so that the major portion of settlement is completed soon after placement of the fill, two approaches may be adopted: surcharging and vertical deep drains, such as sand drains or wick drains.

Surcharging involves applying a load greater than the permanent embankment load for a limited period of time. Figure 5-20 shows a comparison of the compression-time behavior for a cohesive layer due to a permanent embankment load and due to a greater surcharge load. Theoretically, the settlement that takes place in equal time intervals will be an equal percentage of the total settlement for each load. However, the final settlement under the surcharge load will be greater than under the permanent load. After equal time intervals, the settlement under the surcharge will be greater than under the final embankment load. If the surcharge load is applied from time 0 to 0.5T, the compression indicated in Figure 5-20 will be equal to the settlement at 3T under the permanent load. If the extra load of the surcharge is removed, leaving only the final embankment load, the future settlements will proceed as if starting at time 3T under the permanent embankment load. Application of the surcharge load accomplishes in a period of 0.5T what would have required 3T with only the permanent embankment load in place and thus accelerates the time when the permanent track structure can be placed without excessive future settlements.

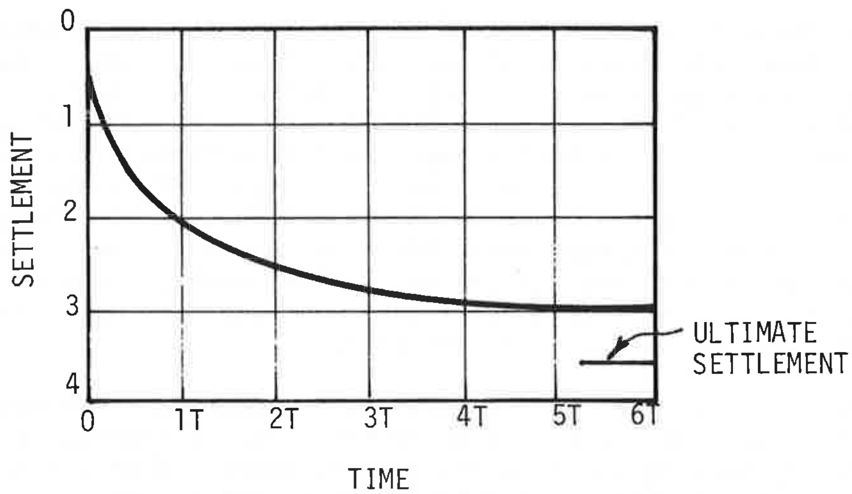


FIGURE 5-19. TIME CURVE FOR TYPICAL LOAD INCREMENT ON CLAY

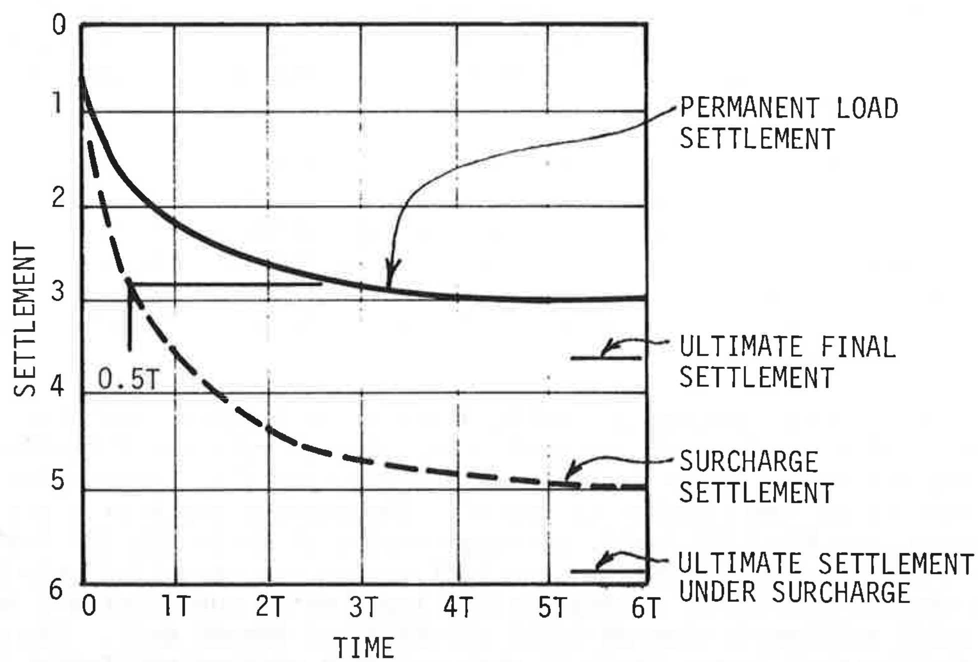


FIGURE 5-20. TIME CURVES FOR SURCHARGE LOADING

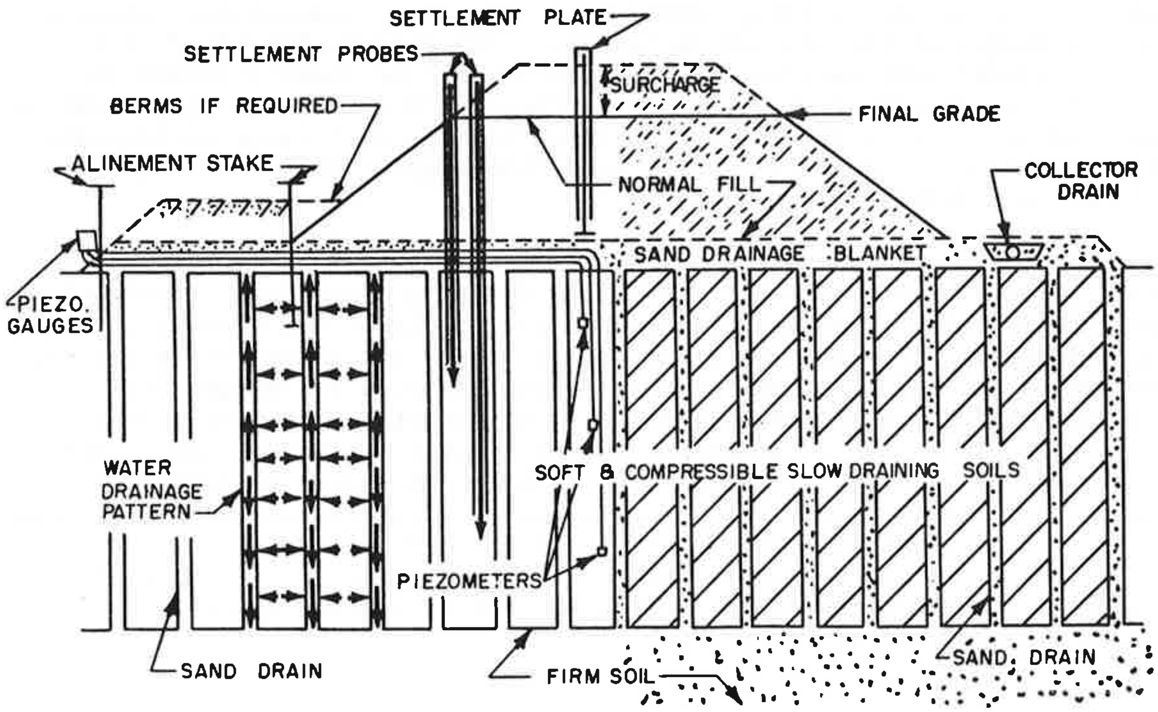
Vertical Drains are water flow paths installed through a cohesive soil layer to accelerate the drainage of pore water from the soil. The drains are installed on a regular grid pattern. A typical cross section beneath an embankment with sand drains is shown in Figure 5-21. Without the drains, water from the center of a cohesive layer must flow through half the thickness of the layer before it can exit the compressing soil. With the drains installed, the water can flow horizontally to the drain and then vertically within the drain, which is many times more permeable than the native soil. In addition, consolidation accelerates because the water flows horizontally through the native soil; the horizontal permeability of most cohesive soils is 2 to 10 times greater than the vertical permeability.

Sand drains may be installed by jetting or augering. The sides of the excavated hole, typically 0.2m to 0.5m (8 inches to 18 inches) in diameter, are maintained by keeping the hole filled with water. When the required depth is excavated, the hole is backfilled with sand to provide the permeable shaft. To prevent clogging by soil fines, the sand gradation must be selected according to filter criteria given in Figure 5-10.

Ten or more years ago, sand drains were installed by driving a mandrel with a flap closure into the ground. Placement was completed by filling the mandrel with the sand and blowing it out with compressed air as the mandrel was withdrawn. This method no longer is considered acceptable, because disturbing the soil by driving the displacement mandrel counteracts the benefits of installing the sand drains.

Wick drains are constructed of filter paper or cloth surrounding an open core. The drains typically are 100mm (4 inches) wide and 5mm to 10mm (0.2 inches to 0.4 inches) thick and are installed using a steel mandrel. The mandrel is pressed into the ground and withdrawn; the wick material is held in the ground by an automatic expanding anchor. Although the mandrel does displace soil as it is inserted, its small size mitigates the amount of disturbance.

The initial spacing of vertical drains is based on analytic studies. Spacing is a function of the soil consolidation rate and the effectiveness of the particular drain installed. Spacing typically ranges from 1m to 5m (3 feet to 16 feet) center to center. The maximum depth of a drain installation is more than 40m (130 feet), although depths of 6m to 10m (20 feet to 30 feet) are typical. Sometimes vertical drains are installed only through the shallow portion of a compressible layer which contributes a greater amount to total settlement than an equal thickness of deeper soil. The compression of shallow soil layers leads to more variable surface settlement than that produced by deeper layers.



Reproduced from Soil Mechanics, Foundations, and Earth Structures, p. 7-6-21, U.S. NAVFAC Design Manual DM-7. Year of first publication: 1971.

FIGURE 5-21. TYPICAL SAND DRAIN INSTALLATION

Engineering - A preload stabilization program requires detailed exploration, laboratory testing, and analytic studies. Undisturbed Shelby tube samples, at least 75mm (3 inches) in diameter, of the cohesive soil should be recovered from test borings. Several consolidation tests should be performed to evaluate the magnitude and rate of consolidation. Analytic studies of expected rate of settlement, the effects and costs of surcharge treatment, and vertical drain installation should be made by an experienced geotechnical engineer. Details of the analysis procedures are beyond the scope of this report, but are outlined in the 1971 U.S. NAVFAC Design Manual DM-7, Soil Mechanics, Foundations, and Earth Structures, Chapter 6. Because the analyses are based on a number of engineering soil parameters that are difficult to determine, full-scale field tests of preload stabilization may be required for major projects. At least some monitoring of settlement progress and pore pressure dissipation during the preload period is required to confirm design assumptions about consolidation rates.

Besides consolidation, the stability of embankments on soft foundation soils must also be considered. Stability is discussed in more detail in Section 5.5, and is particularly important in surcharge evaluations, because the shear strength of the foundation may limit the maximum height of fill that can be placed over an area. If greater foundation shear strength is required, the surcharge may be applied in stages with periods of consolidation between stages. Along with settlement, consolidation will cause an increase in the shear strength of the clay that will then support the next stage of the fill without failure.

Cost - The cost of preloading simply by delaying final track construction is a combination of the value of delaying availability of the track plus the extra interest on investment caused by extending the construction period. If surcharging is used, the cost of placing and removing the excess fill must be considered. Surcharge fill supports no structure but merely adds weight; little compaction is required and spoil materials may be used. The placement and removal cost is about \$3.00 to \$4.00 per cubic yard.

Installing sand drains costs about \$3.00 to \$6.00 per vertical linear foot, but this cost is affected by the required depth and availability of suitable sand to use in the drains. Wick drain installation costs about \$.80 to \$1.50 per vertical linear foot.

Discussion - Preloading, surcharging, and using vertical drains are often the most economical means to treat problems associated with excessive settlement of embankments due to consolidation of foundation soils. The primary limitation is that time is required for consolidation, although the time can be shortened by surcharging or using vertical drains. Predictions of future consolidation are subject to some uncertainty, even though such predictions are routine for an experienced geotechnical engineer. Where refined answers are required, field test sections may be constructed and evaluated to predict prototype performance. Final judgments should always be based on field monitoring of settlements of the actual structures.

Most aspects of preloading must be accomplished during new construction. It may be possible to install vertical drains with the track structure in place. Wick drain mandrels have limited penetration capability; preaugering would be required to install them through the ballast bed or a thick granular embankment. Once installed, the drains also might serve to drain surface water to maintain the performance of near-surface subgrade soils as described in Section 5.1. A more complete discussion of preloading to treat compressible soils can be found in the 1970 papers by S. J. Johnson, "Precompression for Improving Foundation Soils" and "Precompression with Vertical Sand Drains."

Prewetting

Application - Prewetting is specifically applicable to treating clays that shrink and swell due to changing water content. Identification of swelling clays is described in Table 2-4.

Description - Active clays--i.e., clays that exhibit large swelling potential--frequently exist in a dessicated condition. That is, there are surface cracks that form when the clay dries and shrinks. These cracks are the paths that permit surface water to seep rapidly down into the clay, causing a heave. Prewetting permits the clay to swell and heave prior to placing the track structure. The most frequent way to accomplish this is by forming a pond at the surface and permitting the water to seep vertically into the clay. This process may take years unless the clay has a well-developed fissure system or unless wells, similar to vertical drains, are installed to carry water to the deeper soil. The typical time required for prewetting is a few weeks to three or four months(1).

After prewetting, the clay moisture content should be kept constant. Plastic membranes or asphalt sprays may be used, as described in Section 5.3. In this case, the membrane will prevent evaporation and will prevent a downward movement of surface infiltration. Even if the water content of the clay is not stabilized, the strains induced in the clay due to moisture changes after prewetting are smaller than they would be if the clay experienced the same moisture changes without prewetting, because the wetting process reduces the activity of the clay minerals.

Engineering - The principal factors for evaluating a prewetting program are (1) the depth of soil that requires treatment, (2) the vertical permeability of the soil, and (3) the time required to saturate the soil. The maximum depth to be treated should include all partially saturated soil above the water table. Short of that, the depth of soil that actively contributes to swell may be evaluated using data derived from twin oedometer tests, as described in Section 2.3. If the clay is deep enough, the confining pressure will be great enough to prevent swell, even if the clay becomes saturated.

The nature of a vertical fissure system can only be evaluated with test pits and bulldozer cuts that permit direct observation of the fissures with depth. Ponding a small area prior to excavating the pit will permit direct observation of vertical seepage after a given time period.

Field experience will provide the best estimate of the time required to saturate the soil. It may be possible to use consolidation theory to make analytic predictions of prewetting times. Solutions for sand drain

---

(1)G.J. Gromko, "Review of Expansive Soils," Journal of the Geotechnical Division, ASCE, Vol. 100, No. GT6, June 1974, p. 678.

consolidation may be used if vertical drains are used and may even be adapted when fissures provide vertical drainage by adopting an equivalent drain diameter and spacing. The coefficient of consolidation rate should be evaluated from the rebound portion of the free swell test, since prewetting leads to swelling of the clay.

To evaluate the progress of prewetting in the field, soil samples should be taken, and moisture contents should be measured. Careful drilling techniques are required to avoid wetting samples with drill water. Sample water contents should be measured in the field. If testing must be done in a laboratory, special care should be taken to avoid moisture changes of samples during transport.

Cost - The cost of ponding is approximately $0.50 to $1.00 per square yard. If water is scarce, the cost will increase. In addition to prewetting, lime stabilization of the surface layer is typically required to provide a workable surface for operation of construction equipment. The additional costs of lime stabilization and a membrane cover are discussed in Sections 5.3 and 5.4.

Discussion - Prewetting is a cost-effective means of treating a thick stratum of actively swelling clay. However, lime stabilization and using a membrane cover to prevent evaporation from the surface after wetting adds significantly to the total cost. If only a thin layer of swelling soil must be treated, admixture stabilization or replacement of the soil is probably preferable.

Prewetting requires time for the water to penetrate to the required depth. The progress of the saturation may vary and must be monitored by explorations. After the clay is wet, it will have a low strength. Admixture stabilization can alleviate the low strength problem over a limited depth. The track structure must be designed to accommodate the low-strength, saturated clay subgrade so that soft subgrade problems are prevented.

<u>Salting</u>

Application - In-place salting of track is used to treat frost heaving. The intent is to reduce the amount of shimming beneath rails during the freezing season and shim removal during thaw that is required to maintain track geometry. Identification of frost susceptible soils is discussed in Section 2.3. Further discussion of measures to limit frost heaving is included in Section 5.3.

Description - Salt may be applied to the ballast in order to limit the formation of ice lenses. The salt works with rain by creating a brine solution that percolates down into frost susceptible subballast and subgrade and thereby lowers the freezing point of water in the soil, limiting ice formation. The salt may be applied as brine solution. However, this has been reported to be less effective than granular rock salt applied either at the bottom of the cribs or simply on the surface of the ballast section.

In 1972, F. L. Peckover reported in "Frost Heaving of Track - Causes and Cures," that surface application of granular rock salt was the least expensive and most effective method of treating frost heaves. This finding was based on field observations of several methods of chemically treating frost-heave-affected track. Based on field experiments on the Canadian National Railway, application of a 6mm (1/4 inch) layer of salt on the surface of the ballast gave optimum reduction in track geometry deterioration as measured by thickness of shimming required. A 6mm layer corresponds to about 11 kg (25 pounds) of salt per crib. If more salt than this was used, the track became soft during the winter. With the optimum salt application, ice lens formation was prevented. The treatment is not permanent; reapplication is required at 2-year intervals, approximately.

It is possible to apply salt at the base of the crib. Treatment in this manner may be effective for as long as 4 years. This method of treatment was judged less desirable because (1) excavation of the crib to place the salt leads to deterioration of track lateral support, and (2) the salt at depth attracts water both winter and summer, leading to soft track.

With surface application of the salt, Peckover recommends several factors be satisfied, including:

a. Drainage must be adequate <u>before</u> salt is applied.

b. Salt should be applied at a time such that 75mm to 150mm (3 inches to 6 inches) of rain will fall before ground freezing occurs in order to allow percolation of the brine solution into the subgrade.

c. In signaled territory, salt should not be applied to a track length greater than 35m (120 feet) in any one signal circuit.

d. The treatment areas should be terminated with a transition zone about 5m (15 feet) long, where salt is applied at one-half the standard application rate.

Engineering - The principal engineering task is to identify the track sections where salt application is required. Survey procedures are given by Peckover and in the 1968 Russian railway publication, "Instructions on the Straightening of Railway Track at Frost Heaving Sites." Surveys are required both to plan treatment and to evaluate treatment effectiveness. The surveys are based essentially on observations of track geometry that identify areas where active heaving occurs. Evaluation procedures may lead to modification of the rate of application of the salt or to altering the time period for reapplication.

Cost - The cost of salt application is comprised of the surveys, purchase of salt, and labor for distribution. The simplest method of distribution is to unload 22kg (50-pound) bags of salt at the locations to be treated and then distribute them by hand. Salt costs about $130.00 per tonne ($6.00 per hundredweight) or about $1.50 per crib. Installation costs about $0.25 to $0.50 per crib. Total cost of applying salt is about $1.00 to $1.20 per track foot treated.

Discussion - Salt application for treating frost heaves is advantageous because it (1) is inexpensive, (2) is nondisruptive to existing track, (3) can be modified based on post-treatment observations, (4) requires no special equipment, and because (5) experience on the CNR shows the method to be 70 percent effective in reducing shimming required for frost heaves. The disadvantages are that it (1) can interfere with signals if proper precautions are overlooked, (2) promotes corrosion of hardware, (3) requires periodic retreatment, and (4) may pollute groundwater.

Electrochemical Stabilization

Application - Electrochemical stabilization can be used to strengthen and reduce swelling potential of active clay soils.

Description - If electrodes are inserted into the ground and a direct current voltage is applied, electrolyte (water with dissolved salts) will flow from anode to cathode. This process can be used to distribute the electrolyte through the soil. In low permeability soils, such as clays and silts, it is possible to move solutions by electro-osmosis much more rapidly than by hydraulic pressure, as in permeation grouting.

Active clays typically contain minerals with weakly bonded, high exchange capacity cations such as sodium or lithium (e.g. a notoriously troublesome active clay mineral is sodium montmorillinite, called bentonite). If the clay is exposed to solutions containing cations with stronger bonding strength, such as calcium or potassium, the stronger cations will replace the weak cations in the clay minerals. The shear strength of the clay will be increased, and the swell potential will be reduced.

In a typical electrochemical stabilization system, anodes, consisting of perforated iron or aluminum pipe, are installed about 1m to 5m (3 feet to 15 feet) away from the cathodes. A solution, such as calcium, potassium, magnesium, or aluminum chloride, is fed into the anodes where it enters the soil through the perforated pipes. The solutions move toward the cathodes. Water collects at the cathodes and should be removed with a wellpoint system. In addition to injection of the electrolyte solution, the anodes themselves will be electrolytically dissolved, forming iron or aluminum ions. Therefore, the anodes are gradually used up. The cathodes do not dissolve and can be recovered unaffected.

To activate the system, a direct-current electrical source is attached to the electrodes. According to I.W. Farmer in the 1975 book, Methods of Treatment of Unstable Ground (F.G. Bell, ed.), typical voltage gradients to treat silts and clays range from 20 to 60 volts per meter. High currents are required since current directly affects the rate of ion movement. The electrical gradient should follow the hydraulic gradient. Some studies report a 60 percent increase in shear strength and almost complete removal of swell potential by electrochemical stabilization.

Engineering - The specific engineering effort required for electrochemical stabilization includes identificaion of the subgrade soil as a swelling clay. Atterberg limits and hydrometer particle size analysis may be helpful in this effort, as suggested in Table 2-4. Selection of anode type and electrolyte solution is determined by the chemical composition of the clay minerals. X-ray diffraction or differential thermal analysis may be used to identify the minerals. Clay mineralogists may be consulted to assist in the effort.

To evaluate the thickness of soil to be treated, an evaluation of the zone of active swelling is required. This may be done based on laboratory tests, such as twin oedometer tests, to determine how much confining stress is required to confine the swell. Alternatively, installation of settlement observation points at different depths can be used to observe vertical movement versus depth over the course of wet and dry cycles.

The spacing of electrodes, required electrical voltage and current, the time required to achieve results, and the magnitude of soil improvement are all subject to uncertainty. Final design should generally be confirmed by a full-scale field test. Production stabilization should also be monitored in the field. Such monitoring might include sampling and testing of Atterberg limits, laboratory shear strength, or in-situ shear strength. All work should be planned by a geotechnical engineer experienced in soil behavior and soil modification.

Cost - The cost of electrochemical stablization is comprised of engineering, installation of electrodes, cost of the anodes which are expended, cost of operating pumps at the cathodes, and cost of electrical power. Power consumption is high. The cost of all factors combined is approximately $60 to $100 per cubic meter of soil treated. To treat a layer of soil three feet deep and twelve feet wide beneath a track would cost about $250 to $400 per track foot.

Discussion - The principal advantage of electrochemical stabilization is that it could be accomplished without disrupting the track or train operations. The electrodes can be installed on either side of the ties, and operation of the system will not interfere with trains. The principal disadvantages are the extraordinarily high cost and uncertain results. As stated by Farmer, "....electrochemical stabilization methods would be expected to be limited to applications where they are the only feasible method and where the results obtained could justify high operational cost." The method has been successfully used to treat an expansive clay beneath an in-service Arizona highway, as reported in 1976 by C. E. O'Bannon et al. in "Electrochemical Hardening of Expansive Clays."

5.3 LAYER INSERTS

Layer inserts comprise manmade or natural materials that are installed in the substructure to stabilize or prevent deterioration of track substructure performance. Most inserts are used to prevent problems that are caused by poor subgrade behavior and are placed either directly beneath the top ballast or deeper into the substructure. Descriptions of layer inserts suitable for track stabilization are presented in the following subsections.

Subballast

Application - Subballast is placed in the track substructure to improve track performance by limiting stresses on the subgrade, preventing the mixing of the subgrade and ballast, draining surface water to the sides of the track to limit moisture in the subgrade, and reducing frost penetration into the subgrade.

Subballast improves performance of track placed on all soils. When properly graded to provide filtering capabilities, subballast is most beneficial to lower-quality subgrades--such as fine sands, silts, and clays--that tend to pump and to foul the ballast if no protective layer is provided.

Description - Subballast materials were previously discussed in Section 4. Typically, well-graded crushed stone, sand and gravel, or admixture-stabilized aggregates are used. Selecting appropriate subballast was discussed in Section 4.3.

Cost - The cost of subballast compacted in place is about \$8.00 to \$15.00 per cubic yard, depending on material availability. To provide a 12-inch subballast bed, 15 feet wide, costs about \$4.50 to \$8.30 per track foot. Therefore, the premium cost in new embankment construction may not be as high. Extra excavation is required for using subballast in cuts, thereby raising its installation cost.

Discussion - Subballast should always be considered in cases where track will be removed and there is space to install the material. Subballast has one of the best cost-benefit ratios of all stabilization measures. To limit subgrade stresses, it is preferable to add subballast rather than increase ballast thickness, because subballast has higher vertical and lateral stability than ballast and usually costs less. However, when it is necessary to correct pumping problems experienced by in-service track, the measures discussed below generally are more economical--unless complete reconstruction of the line is justified.

Filter Fabric

Application - Filter fabric--also called filter cloth or geotextiles--is made from plastic fibers. Porous and permeable to water, with a finite opening size that restricts the passage of solid particles, filter fabric separates earth materials of different particle sizes yet permits flow of water. It has been used in railroad track to prevent mixing of ballast with the underlying subgrade, which results in mudpumping, ballast pockets, and fouled ballast. These problems are most prevalent in silty and clayey soils (USC classes with C or M symbols) and fine, sandy soils (SP and SM). Fabrics also may be used instead of graded filter underneath riprap (stone blanket) slope protection to prevent piping and erosion of fine soil through the riprap which will destroy its foundation. Filter fabrics also have been used in underdrains, where fabric replaces graded filter aggregates surrounding a drain pipe, as discussed in Section 5.1.

Description - Filter fabrics typically are made of plastic--such as polypropylene, polyester, polyethylene, nylon, or a combination of these. Each of these materials provides different properties with respect to modulus, strength, creep, and resistance to rot and chemicals. Limited service data are available for selecting filter material. However, it is known that, generally, material performance and cost will be directly related.

The first fabrics produced about 20 years ago were manufactured by weaving and bonding the woven fibers to maintain filament positions. Nonwoven fabrics were developed 6 to 10 years ago; these are produced by spraying out multiple layers of filaments. The filament mat is then subjected to heat or resins so that intersecting filaments bond together to form a stable fabric. Several bonding methods are used, including adhesive bonding, heat bonding, and hot needle punching.

Fabrics weigh from one-half ounce to more than 20 ounces per square yard (13 to 50 grams per square metre) and have thicknesses of 0.01 inches to 0.2 inches (0.25mm to 5mm). The thickness-weight relation varies for the different types of fabrics and for different manufacturers. Heat-bonded fabrics generally are thinner for a given fabric weight, so that longer rolls may be unrolled beneath the limited space provided by a track sled or undercutter. Needlepunch fabrics usually are thicker, with a texture similar to felt. According to Jack Newby in the 1980 paper, "In-depth View of Geotextile Subgrade Stabilization," the needlepunch fabrics appear to be less prone to plugging by soil fines and permit greater lateral flow of water within the fabric, a significant factor. Definitive criteria on fabrics for railroad applications have not been developed. As reported by Gordon Benson in the 1978 report, "Interim Guide Specifications for Filter Fabrics," most filter fabric applications in railroad track have used fabrics weighing about 4 ounces per square yard (50 g/sq. m). Heavier weight fabrics also have been used effectively according to Newby.

In the past six years, filter fabric has been used increasingly for stabilizing subgrades as part of track maintenance and rehabilitation programs. Many of these filter fabric projects have received attention in railroad industry magazines and journals. Research programs on in-service performance of fabrics have been sponsored by industry(1) and by the Association of American Railroads(2). Most fabrics have been used for track problem spots associated with high dynamic loads including grade crossings, switches, frogs, and other special track. Some sections of regular track with fouled ballast and other continual maintenance problems have been rehabilitated using fabric.

When used in track, filter fabric can separate aggregate such as ballast from underlying subsoil; permit movement of groundwater while preventing movement of soil particles; provide a tensile reinforcement layer to enhance mechanical performance of the substructure; and drain the near-surface subgrade soil, thereby stabilizing water content.

Filter fabric can be placed either directly beneath the ballast or (less frequently) between the subballast and the subgrade(3). There are arguments in favor of each of these locations. When placed beneath a subballast layer, the filter fabric prevents migration of fines from contaminating the subballast. Such a design may be desirable in place of the two-layer subballast system previously described in Section 4.3. Subballast typically is well-graded, and vertical stresses are limited so that puncturing the fabric is unlikely.

However, when fabric is installed as a rehabilitation measure with undercutting or ballast sledding, it is impractical to place a subballast over the fabric. The fabric is rolled out directly behind the undercutting chain or plow blade, so that the fabric rests on a layer of mixed ballast and subgrade soil. The cleaned or fresh top ballast is then dumped directly on the filter fabric. Coarse, angular ballast particles can puncture the fabric, thereby limiting its effectiveness. Therefore, filter fabric should exhibit high tensile strength, large elongation before failure, and high puncture resistance. Heavier-weight fabrics (6 ounces to 10 ounces per square yard) are sometimes used to prevent puncturing. Fabric installations have been investigated

---

(1)"Engineering Fabrics...Used and Researched by Southern Pacific," Progressive Railroading, Vol. 22, No. 3, 1979, pp. 52-58 (in particular, the Caldwell Texas test section by Monsanto).
(2) T.A. Haliburton, "Use of Geotechnical Fabric in Railroad Operations, Report No. R-456, August 1980, Association of American Railroads.
(3)"Stabilization Fabrics: Where They Are in Railroading Today, Railway Track and Structures, No. 855, June 1978, pp. 42-44.

in a number of locations throughout the United States and Europe. Some failures have been observed. However, puncturing has not been a problem if sufficient ballast is placed between the bottom of the ties and the fabric, and tamping equipment does not puncture the fabric.

Even for new construction, it may be advantageous to place the fabric at the base of the top ballast bed because the fabric will improve the mechanical performance of the ballast. The fabric's tensile strength interacts with the ballast by friction and imparts a lateral confinement to the ballast that, in turn, increases the ballast strength and stiffness. The load applied by the ties is spread over a wider area, thus reducing ballast pressure on the subgrade and improving subgrade performance. Regardless of the position of the fabric, it must be completely covered after installation because exposure to sunlight will degrade its plastic materials.

Manufacturers of filter fabrics claim that drainage of the near-surface subgrade soil is accomplished by two actions. First, the fabric preserves the high permeability of the ballast at the base of the bed so that lateral drainage of surface water occurs rapidly, and less surface water gets into the subgrade. The weight of the ballast bed and track structure is thus more effective in consolidating and stabilizing the shallow portions of the subgrade. Second, manufacturers of the thicker, needlepunched fabrics or felts claim that the thickness of these materials enables them to move moisture laterally to the edge of the fabric by capillary action and gravity or siphon flow sometimes called a "wick effect." Field evidence of this drainage mechanism has yet to be positively confirmed. Studies of filter fabric installations in railroad track demonstrate the ability of fabrics to perform the separation and filtering functions. However, the lateral confinement-load spreading action is difficult to determine. Field experiments are underway to study this factor(1).

The fabric's greatest strength is required beneath the ties with a separation capability still beneficial beneath the shoulders. Special fabrics have been produced for railroad use that are heavier in the center than near the edges. A similar effect can be produced using a double layer of fabric that is overlapped under the ties and yet has a single layer in the shoulder regions.

Engineering - The principal function fabric performs is to limit ballast fouling. However, before choosing a filter fabric to correct the problem, it is important to identify the source of ballast contaminants. Sampling and testing of ballast in-track, as described in Section 3.3, and corresponding sampling and tests of the underlying subgrade soil, described in Section 2.6, should provide data necessary to identify the source of fines in ballast. Assuming fines are moved from the subgrade into the ballast, a filter fabric can correct the fouling problems.

---

(1)"Engineering Fabrics..." Progressive Railroading, Vol. 22, No. 3, p. 52.

Specification of fabric properties to be used in track is not well established. In the 1978, "Interim Guide Specifications for Filter Fabrics," by G. R. Benson, the following criteria were suggested for filter fabrics used at railroad crossings and may be suitable for general track applications. These criteria are based on a consensus of parameters from successful fabric installations.

1. Weight of fabric (ASTM D1910): greater than 4 ounces per square yard (greater than 135 g/sq.m).

2. Grab tensile strength (ASTM D1682): greater than 90 pounds (greater than 400N).

3. Elongation at break (ASTM D1682): greater than 25 percent.

4. Equivalent opening size (EOS): No. 70 mesh.

The U.S. Department of the Army, Corps of Engineers, guide specification for "Plastic Filter Fabric" (CW02215 of November 1977) provides two criteria for selecting fabric according to particle-holding capability and water permeability. The EOS of a fabric is the size of U.S. standard sieve that most closely matches the opening size of the fabric. The guide criterion is that the EOS should be less than the D_{85} of the finer material (the sieve size through which 85 percent of the material passes) if the material has less than 50 percent fines; otherwise, the EOS should be no coarser than No. 70 sieve and no finer than No. 100 sieve. Fabrics should not be used alone to protect soils with more than 85 percent finer than the No. 200 sieve.

Permeability is measured by the gradient ratio that considers the permeability of the soil being protected. The fabric is placed in a constant head permeameter (see Figure 2-6) supported by a 1/4-inch wire cloth. A soil specimen 4 inches long is placed above the fabric. Piezometer taps are installed 1 inch below and 1 inch, 2 inches, and 3 inches above the fabric. The gradient ratio is the quotient of the hydraulic gradient over the fabric and the 1 inch of soil above the fabric divided by the gradient over the 2 inches of soil between 1 inch and 3 inches above the fabric. The gradient ratio should not exceed 3.

Cost - The cost of 4-ounce, nonwoven fabrics is about \$0.08 to \$0.10 per square foot; the cost of woven and heavier nonwoven fabrics is about \$0.30 to \$0.50 per square foot. Fabrics are supplied in widths of 6 feet to 30 feet. Installation labor is minor, whether rolled out by two men on an exposed subgrade or automatically unrolled beneath an undercutter or sled, and costs about \$0.50 per running foot. Assuming that the fabric is supplied in a 15-foot-wide roll, the cost of installing fabric in association with other construction or maintenance operations is \$1.70 to \$2.25 per track foot.

Discussion - Although its use has been limited during the past 6 years, filter fabric has provided positive separation of ballast and subgrade soil to reduce fouled ballast, improve drainage, stabilize moisture content of swelling clays, and limit ballast pockets and other progressive soft subgrade problems in many installations. It has also been claimed to improve the mechanical performance of ballast. Filter fabric is convenient and easy to install as part of rehabilitation measures such as ballast undercutting or sledding. Special equipment is not usually required. However, J. Newby reported that a crane is required to provide sufficient clearance for the fabric roll behind a ballast undercutter. Because the material is manufactured, field control problems are limited to ensuring a proper overlap between pieces (usually 1 foot to 3 feet) and sufficient ballast depth below ties to avoid puncturing or overloading.

The principal difficulties with filter fabric installations have arisen when there is insufficient ballast between the base of the ties and the fabric. There must be at least 0.15m (6 inches) of ballast below the ties before tamping can begin. Practices in Germany consider 0.2m (8 inches) as the minimum cover over the fabric. Although 0.15m (6 inches) is the minimum ballast depth recommended, greater permanence can be expected if the final depth of ballast below the ties is closer to 0.3m (12 inches).

Because of the recent adoption of fabrics, it is not possible to predict the useful life of the material in track. Fabrics have performed satisfactorily in drainage structures for as long as 20 years. If installed with proper attention to ballast cover and with generally good maintenance practice--especially proper drainage--filter fabric can be expected to have a long life in track.

Impermeable Membranes

Application - Impermeable membranes are used to treat subgrade problems generated by surface water infiltration and evaporation of water, or to interrupt the capillary rise of groundwater necessary to form an ice lens (as discussed later in this section). Surface water infiltration and evaporation are principal factors that activate swelling clays. These are generally CH materials, as identified in Table 2-4. An impermeable membrane can be placed below the subballast to preclude precipitation that would saturate and swell active soils from entering the subgrade. If the swelling soils have been pre-wet, as described in Section 5.2, the membrane may be used to limit evaporation to maintain the high water content.

Impermeable membranes might also be considered to provide a tensile reinforcement layer, as described in this section under "Filter Fabrics," and to limit infiltration into any type of soil that is sensitive to moisture. The problems with this application are discussed below.

Description - Impermeable membranes are solid, flexible plastic sheets which are manufactured from neoprene, polyvinyl chloride, chlorinated polyethylene, or other plastics. If high strength is required, the membrane typically is bonded to a nylon or polyester mesh. Membranes suitable for use below track are available in a range of thicknesses from about 6 mil (0.006 inches, 0.15mm) to at least 120 mil (0.12 inches, 3.0 mm). Section 1.2.5.3 of the AREA Manual suggests a membrane thickness of 1/16 inch (62.5 mil, 1.6mm) for this application, a relatively thick membrane.

To prevent infiltration of water from the surface into the subgrade, the membrane is placed at the top of the subgrade and must be protected from puncturing. Therefore, it must not contact the ballast. The membrane is installed between two blankets of sand 75mm to 150mm (3 inches to 6 inches) thick that may be covered by several additional inches of subballast.

When an impermeable membrane is installed to limit the growth of ice lenses by interrupting the upward flow of water into the freeze zone, it must be placed at a depth that is above the groundwater level yet is below the depth of freezing. This application usually is practicable only when an embankment of frost-susceptible soil is constructed over an area of shallow groundwater. Protection of the membrane against puncturing is still required.

Engineering - In the design of a stabilization system incorporating an impermeable membrane, it is important to recognize that the membrane will block water movement in both directions. Thus, if a membrane is installed to block downward percolation, it must be recognized that it will also stop upward flow that would occur due to surface evaporation. In swelling soils, it is often reduced evaporation due to covering an area that results in heave. However, if the soil has been pre-wet prior to covering, the membrane will limit evaporation and help maintain a stable water content. The soil above the membrane will still receive surface infiltration, and appropriate means must be provided to drain this water, such as by installing the membrane with a crown to promote drainage to the sides of the track. Free-draining fill should be used above the membrane. In designing the crown, note that subgrade settlement will occur under applied embankment load. The crown must be sufficient to avoid dishing of the membrane after settlement is complete.

Cost - Membrane materials cost from $0.10 to $2.00 per square foot depending on the type of material used and its thickness. The 1/16-inch-thick neoprene sheets suggested in the AREA Manual cost about $1.00 to $1.20 per square foot. Installation of the membrane costs about $0.20 per square foot assuming that the membrane is supplied in a sufficient width so that only seams are required at the ends of the rolls. In addition to the membrane, a sand blanket is typically used. This material, installed in two thin layers, costs about $10.00 per cubic yard.

For the purpose of a cost comparison, it is assumed that the membrane would be installed over a width of 15 feet, protected by two sand blankets 4 inches thick. The estimated cost for this membrane protection is $21.00 to $25.00 per track foot.

Discussion - If properly protected from puncture, an impermeable membrane will provide a cutoff of water flow. However, this may cause unintended changes in soil moisture, such as saturation of soil beneath the membrane due to trapped evaporation. Therefore, both the primary and secondary effects of installing a membrane must be considered; a test section may be required to confirm predicted behavior, including observations over a three-year to five-year period.

Membranes may also provide a surface of low shear strength that could contribute to instability. This may be due to slippage between the membrane and soil or due to reduced soil shear strength near the membrane where moisture content is unusually high.

In summary, impermeable membranes are used to block the flow of water that contributes to track subgrade problems. Protection is required against membrane puncturing since any holes may cause local subgrade failures. The membranes may also produce changes in parts of the substructure that are both unintended and undesirable. Since the membrane must be carefully protected, it should be placed deep in the substructure, so that its use is probably limited to new construction or reconstruction. Membranes are expensive, and their success probabilities are unknown at this time. Full-scale field tests are needed for accurate evaluation of success.

Bituminous Spray

Application - Bituminous sprays are applied to the surface of soils and to aggregates to limit infiltration of water and to cement soil particles, thereby limiting the softening of the soil due to increased water content. The spray is either applied at the surface of the subgrade or subballast, or may be used in association with undercutters or sleds during track rehabilitation to treat and stabilize the in-place surface of fouled ballast. Any type of soil can be beneficially treated with a bituminous spray except for coarse gravels (the pores are too large to produce effective treatment) or highly plastic clays (the clay permeability is already low and particle cohesion is already high). Generally, the soil should have a moderate permeability (greater than 0.0001 cm/sec) to permit some penetration of the bitumen into the soil.

Description - Bituminous spraying is the surface application of asphalt emulsions or cutbacks. These are explained more fully in Section 5.5, under "Bituminous Layer Stabilization." This application is similar to bituminous bound or "oiled" gravel roads. The liquid asphalt is applied from a spray bar attached to a tank truck. If applied as part of track maintenance procedures, the spray bar may be attached behind an undercutter chain or plow blade. Ballast may be replaced immediately after asphalt application. The asphalt liquids are applied at a rate of 6.8 to 11.3 liters per square meter (1.5 to 2.5 gallons per square yard). The material is sprayed at a pressure of up to 350kPa (50 psi) to promote penetration.

The bituminous-sprayed layer will limit vertical movement of surface water. Prior to spraying, the soil surface should be graded to drain the water above the stabilized layer. Highly permeable material, such as ballast or clean gravel, placed above the layer will promote lateral drainage of surface water.

Engineering - The general exploration and testing methods described in Sections 2.1 and 3.3 are recommended for identifying areas where bituminous spray stabilization is suitable. Bituminous spray is most effective in stabilizing low-plasticity cohesive soils and silty sands that are softened by water penetrating from the surface. To identify this condition, water content measurements versus depth may be helpful. In-situ shear strength measurements, using a small penetrometer or vane shear apparatus, can provide guidance for planning and evaluating spray stabilization.

As with impermeable plastic membranes, the spray treatment will limit downward percolation as well as upward evaporation of water. This latter factor should be considered to avoid trapped evaporation that might saturate and weaken the soil beneath the stabilized layer.

To design a bituminous spray stabilization project, the viscosity and application rate of the spray must be selected. Permeability tests provide some indication of the penetration of the asphalt. Laboratory test applications may provide some indication of the reduction of permeability provided by asphalt. In general, bituminous spray applications are designed based on previous field experience. Asphalt spray penetration is commonly used in pavement construction. There is some experience in the use of asphalt sprays in railroad substructure. In 1977, T. Ino reported the use of emulsified asphalt as part of a "Reinforced Subgrade," in which the spray was applied to crushed slag. In 1972, D. J. Ayres reported "Unstable Track Formations Respond to Bituminous Spray Treatment"; installed in conjunction with ballast cleaning, the method was reported to reduce mud pumping and soft track.

Cost - Bituminous spray stabilization is only practicable if the surface to be treated is exposed, such as during new construction, ballast undercutting, or sledding. The cost of the spray treatment comprises the asphalt cost plus the application cost. The cost of the asphalt is $0.55 to $0.85 per gallon. Application costs about $0.10 to $0.15 per square yard. To treat a track area 15 feet wide, at a rate of 2 gallons per square yard, the cost is $2.00 to $3.10 per track foot.

Discussion - The advantages of bituminous spray stabilization are as follows:

1. If applied to a fully exposed surface without interference, the equipment is readily available from highway paving contractors.

2. Bituminous spray can be applied with limited equipment modification as part of a ballast undercutting or sledding operation.

3. No protective cover is required.

4. The cost is relatively modest.

The disadvantages of spray treatment are:

1. There is only limited experience using the method on railroad tracks; effectiveness relative to prevention of subgrade softening and mud pumping is uncertain.

2. If the ballast is placed directly on the treated surface of a soft subgrade, ballast intrusion into the layer may significantly increase the treated layer permeability. In general, this method is less effective in cutting off the flow of water than a properly installed plastic membrane.

3. The spray-stabilized layer will trap evaporation that may lead to subgrade softening.

4. Bituminous spray has not been used to limit capillary movement of water during frost heaving. The variation in permeability imparted by the spray may lead to significant track roughness due to differential heaving. However, since the layer would produce a lower average vertical permeability, the average frost heave magnitude, as well as the differential heave magnitude, may be reduced. Field trial is required for proper evaluation of this use of bituminous spray stabilization.

Frost Heave Reduction Measures

Application - Frost heaving is discussed in Section 2.2, and classification of soils with respect to frost heave susceptibility is described in Table 2-3 and Section A6.3. Frost heaving requires penetration of subfreezing temperatures into partially saturated subgrade soil, upward movement of groundwater by capillary flow, and a frost-susceptible soil. Stabilization measures to reduce frost heaving either reduce the depth and rate of frost penetration into frost-susceptible soil, reduce the rate of water movement into the freezing zone, or both. The soil types most susceptible to frost heave action are nonplastic silts, silty fine sands, and low-plasticity cohesive soils (USC classes ML, SM, and CL), as identified in Table 2-3.

Engineering - The Russian railways deal with frost heaving extensively. In 1968, the U.S.S.R. Ministry of Transport issued "Technical Instructions on Treatment of the Roadbed Subgrade in Heaving Sections," which recommends that the following information be determined for planning frost heaving mitigation measures:

a. Location of heaves and sags; profile of heave magnitude determined at the time of maximum heaving.

b. Track characteristics (e.g., superstructure, speed, grade, cross section).

c. Depth of freezing; in pavement practice, this is usually evaluated based on the coldest year in 10 years.

d. Substructure profile (groundwater level, ballast type and condition, drainage, and subgrade soil properties).

The exploration and testing methods described in Sections 2.1 and 2.6 are recommended to gather information on subgrade soil. The procedures described in Section 3.3 are recommended for collecting data on in-track ballast conditions. In the 1972 article, "Frost Heaving of Track-Causes and Cures," F. L. Peckover provides additional details to identify areas in which frost heaving occurs and presents material on corrective actions. Some of these actions are discussed below. When planning for railroad construction, observing pavement and roads in the surrounding area can indicate the potential for frost heaving.

Remove Frost-Susceptible Soil

Description - The most straightforward way to limit frost heaving is to remove all frost-susceptible soil above the depth of maximum frost penetration. This may be accomplished by three means:

1. Excavate susceptible soil and replace with clean granular soil. Based on pavement design practices, only partial protection normally is required (to about 60 percent of the maximum expected frost depth).

2. Raise track, either new or existing, on ballast or fill of nonfrost-susceptible soil; remove fouled ballast; lift track; may require widening of fill.

3. Widen embankment to limit frost penetration from the edges of the track.

Cost - Excavation/replacement are conventional earthwork measures and track maintenance operations. The cost of this work is discussed in Section 5.5 under "Excavation/Replacement."

Discussion - As a factor in the design of new construction, frost-susceptible soils should be avoided in the portion of fills that will remain within the freezing zone. In cuts, a highly frost-susceptible soil and shallow groundwater may indicate that an extra thickness of clean subballast should be provided to limit frost penetration into the subgrade. In existing track, excavation/ replacement will probably be a costly means to correct frost heaving, due to the depth of excavation required. Complete disruption of the track is required to carry this out.

Insulation

Description - Insulating materials may be placed in the substructure to reduce ground heat loss to the air, limit the depth of frost penetration, and reduce the rate of ice lens growth. The insulating layer should be placed above the top of the zone where ice lens formation occurs.

The following materials have been used to insulate subgrades beneath railroad tracks or pavements: (1) rigid foam planks; (2) lumber (ties); (3) asbestos tailings; and (4) peat or bark. Foam typically is polystyrene foam, urethane, or polystyrene that is supplied in bats approximately 0.6m by 1.2m (2 foot by 4 foot) up to 1.2m by 3.6m (4 foot by 12 foot), in thicknesses of 25mm to 102mm (1 to 4 inches). The other materials have all been used for railroad subgrade insulation in Europe but are judged to be of limited application in the United States for the following reasons:

1. Lumber would have to be placed in a position where it would remain damp, resulting in rapid rotting. Even preservative treatment would extend the lumber's lifetime only a short while. Ties of a sufficient quality to provide support would be too costly to be used for this purpose.

2. Asbestos tailings are available in limited quantities. The health hazards of working with asbestos may severely restrict its use.

3. Peat or bark is too soft to provide adequate support for track. Continued rotting of the peat leads to progressive track settlement. Peat and bark have been used on the Norwegian Railways; however, the much higher axle loads in North America make this method unsatisfactory.

The thermal conductivity of soil ranges from about 1 to 30 calories per metre per hour per degree Celsius. The thermal conductivity of insulating materials such as foam bats is about 0.05 to 0.20 Cal/(m)(hr)(°C). Therefore, 50mm of foam provides protection against frost damage similar to that of 0.5m to 1.0m of soil cover. Even if the insulation does not totally prevent formation of ice lenses, it will slow the rate of heat loss from the ground and reduce the thickness of the ice that forms.

Foam boards may be manufactured with interlocking edge joints. Thus, the boards will form a partially impermeable layer, shutting off water as well as heat flow. They must be anchored or joined to keep the joints from opening.

Engineering - In addition to general engineering factors mentioned under "Frost Heave Limitation Measures," insulation stabilization requires an evaluation of the thickness and lateral dimension of the insulation. The principle of insulation design to prevent frost heaving is to keep the freezing line (0°C line) above any frost-susceptible soil and to limit the rate of heat loss from the ground which determines ice lens growth rate. Design of insulation thickness may be based on heat flow equations presented in the 1966 textbook by A. R. Jumikis, Thermal Soil Mechanics. Approximate thickness design may be based on the relation that 25mm (1 inch) of foam insulation will reduce frost penetration in soil by 0.4m (15 inches). The design depth of frost penetration may be based on data provided in Sections A6.1 and A6.2 or locally available data. Depending on the cost of the installation and the track geometry criteria to be maintained, the design may provide either complete or partial protection against frost penetration into susceptible soil.

In the 1975 paper, "Insulated Road Study," E. Penner showed that frost penetration from the edge of an insulated pavement progressed as far laterally as frost would penetrate below an uninsulated area. Insulation must be provided over a sufficient width and should be tapered at the sides and ends of the stabilized section to avoid an abrupt change of surface. As a final design verification, a test section including temperature measurements and heave observations of insulated and uninsulated track sections is recommended.

The final design factor is to determine the thickness of cover over the insulation to prevent track stresses from damaging the materials due to cracking or compressing out the voids. Rigid foams have compressive strengths of 140 kPa to 280 kPa (20 psi to 40 psi). The stronger, denser foams have higher thermal conductivity rates (less insulation capacity). The static stress of the track structure and substructure up to several feet deep is less than 35 kPa (5 psi). Therefore, the principal objective is to limit dynamic stress due to train loads. Conventional stress analysis methods for analyzing track may be used. These would indicate that the insulation should be placed about 0.4m to 0.8m (15 to 30 inches) below the base of the ties. Insulation should be separated from coarse rock ballast by a blanket of sand, or sand and gravel. The insulation panels should be set on a subgrade surface that is carefully graded to drain so that uniform support is provided, and water above the insulation will flow to the shoulders. Therefore, insulation can only be installed practically in new construction or reconstruction.

Cost - The material cost of board insulation is about $0.20 to $0.30 per square foot per inch thickness. Installation costs about $0.10 to $0.15 per square foot.

Assume insulation were to be selected to protect a subgrade where the frost depth is 5 feet and the ballast bed is about 14 feet wide at the base. To counter frost penetration at the edges, the insulation width must be about 20 feet. The cost for installation of 2 inches of foam insulation is $10.00 to $15.00 per track foot.

Discussion - The advantage of using insulation to limit frost heave is that it can be placed at a shallow level in the substructure. (Water flow cutoffs must be placed below the maximum freezing level.) There has been some successful experience in highways using foam board insulation, and the use of asbestos tailings and peat insulation is reported on Russian and Norwegian railroads. However, no experience in using insulation on North American railroads was discovered. Insulation can only be placed during new construction or, if sufficient substructure can be placed above the insulation, during rehabilitation.

Capillary Interrupt

Description - Creation of an ice lens in frost-susceptible soil requires water to be drawn upward from the groundwater table to the zone of freezing. The movement of the water is caused by capillary or surface tension forces in the zone of unsaturated soil. To cut off this upward movement of water, a layer of coarse soil is placed in the subgrade. The "interrupt layer" must be situated above the winter groundwater table and below the zone of freezing, as shown in Figure 5-20.

The capillary rise of water in soil is governed by the size of the pores in the soil. As discussed in Section 2.2, pore sizes in soil are related to particle size distribution. Table 5-4 indicates the height of capillary rise for various types of soil and their respective D_{10} particle sizes. To be totally effective, a sufficient thickness of coarse soil should be installed to keep the capillary rise from crossing the layer. As shown in Table 5-4, the coarser the material, the thinner the layer. However, a gross difference in particle gradation between the interrupt layer and the surrounding soil may lead to piping of soil fines into the layer. This will significantly decrease its effectiveness with respect to capillary interrupt and drainage. Even if the full thickness of material is not provided, installation of a partially effective interrupt layer will reduce the rate of flow and limit the thickness of ice lenses that form. Laboratory tests of capillary rise may be performed, as described in 1946 by Lane and Washburn in "Capillarity Tests by Capillarimeter and by Soil Filled Tubes."

The location of the groundwater table and its variation during the freezing season are essential factors in evaluating the proper depth of the interrupt layer. Observation wells should be installed and monitored during the winter months to determine water levels during the freezing season.

TABLE 5-4. HEIGHT OF CAPILLARY RISE

Soil	D_{10} (mm)	Capillary Rise (mm)	Capillary Rise (ft)
Coarse Gravel	0.82	0.05-0.06	0.16-.2
Fine Gravel	0.30	0.1-0.2	0.3-0.7
Sandy Gravel	0.20	0.2-0.3	0.7-1.0
Coarse Sand	0.11	0.6-0.8	2.0-2.6
Silty Gravel*	0.06	0.7-1.1	2.3-3.6
Fine to Medium Sand*	0.02	1.2 -2.4	3.9-7.9

*Unsuitable material for capillary interrupt.

Source: K. S. Lane and D. E. Washburn, "Capillarity Tests by Capillarimeter and by Soil Filled Tubes," Proceedings, Highway Research Board, 1946.

Cost - Installation of a capillary interrupt layer would normally be included as part of costs for new construction or for complete reconstruction. The interrupt layer typically replaces general fill in the embankment. As such, the cost of the interrupt layer is the premium cost for the coarse-grained materials, usually ranging from $2.00 to $4.00 per cubic yard. The placement costs and most other elements would be required with or without the installation of special material. In a cut area, extra excavation would be required to install the interrupt layer. In existing track, installation would require excavation of existing fill. Excavating and replacing frost-susceptible soil would be more economical than installing an interrupt layer in these cases.

Since the interrupt layer is installed deep into the subgrade, the width of the layer must be increased to underlay the entire embankment. Assuming a 25-foot-wide, 1-foot-thick layer would be installed, the cost of treatment is $2.00 to $4.00 per track foot as part of fill construction.

Discussion - The advantages of a capillary interrupt are:

1. Conventional construction methods may be employed.

2. In addition to limiting capillary rise and frost heaving, the coarse layer will also serve as a blanket drain to remove water from the embankment; however, unless properly protected by filters, gravity flow of water into the layer can carry fines into the material which will diminish its capacity for capillary interrupt and drainage.

3. The cost is modest; not figured in the cost factors discussed previously is the consideration that incorporation of a capillary interrupt layer may permit use of frost-susceptible soils above the layer that might otherwise lead to severe frost heaving if left unprotected; this may lead to cost advantages that outweigh the cost of using a small quantity of select material.

The major disadvantages of a capillary interrupt are:

1. It is only applicable to new construction or complete reconstruction.

2. If the distance between the maximum frost depth and the water table is small, this method expands to excavation and replacement of all frost-susceptible material in the freezing zone.

3. Reliable procedures for selecting the thickness required for complete protection have not been established.

4. Fine particles may contaminate the coarse materials, destroying their effectiveness as an interrupt layer.

Clay Blanket

Description - A capillary interrupt layer placed in the subgrade forms a barrier across which capillary flow is impeded. A clay blanket will support a very large capillary rise; however, the permeability of clay is very low. Thus, the clay blanket merely slows the flow of water. Thus, the thickness of the potential ice lens is limited by the quantity of water that can cross the clay blanket during a freezing season.

Clays are notorious for their poor performance as railroad subgrades. The clay blanket should be placed deep enough so that dynamic stresses from train loading are limited. The blanket must be properly graded for drainage to avoid softening the clay. Clayey gravels are used typically so that the layer will have both high strength and low permeability. Swelling clay should be avoided.

Cost - Cost factors for a clay blanket are similar to those for the capillary interrupt. Premium costs for material are estimated to be from $2.00 to $4.00 per cubic yard. The cost per track foot is $2.00 to $4.00.

Discussion - Clay blankets are installed by conventional methods, and the costs are moderate. Clay blanket use is only practical for new construction or reconstruction. The principal concern is to avoid placing the clay blanket in a position where it will contribute to soft track or embankment instability.

5.4 COMPACTION AND ADMIXTURES

Compaction can densify shallow subgrade soil that is too loose to satisfactorily support the track structure by rapidly reducing the volume of voids. Compaction does not include methods that induce time-dependent consolidation of the soil by quasi-static loading, as described in Section 5.2 under "Preloading."

Compaction methods are used to stabilize granular soils (USC codes GW, GP, SW, SP) and granular soils with fines (GM, GC, SM, SC). Low to moderately plastic cohesive soils (ML, CL) and highly plastic soils (MH, CH) can only be compacted if they are partially saturated. Therefore, if plastic soils are saturated, they must first be dried to become partially saturated, or they must be consolidated by applying a static load for a period of time. (Consolidation stabilization is discussed in Section 5.2 under "Preloading." Several methods of in-place compaction of deep soil layers are also described in Section 5.2 under "Deep Densification.") Organic soils (OL, OH, Pt) generally cannot be improved by compaction; instead, removal or preloading is advised.

Many soils can be compacted effectively in their natural states or by adjusting their water contents to facilitate compaction. Some soils, however, cannot be compacted in their natural states to produce a stable railroad track subgrade that will perform satisfactorily when subjected to train loads. These may include uniformly graded sands (single-sized particles), nonplastic silts, and slightly plastic and very highly plastic clays. In such cases, it may be best to excavate and replace the soil, or to introduce admixtures into the soil to improve its response to compaction and to improve its in-service performance.

Compaction

Application - Compaction methods are used to place soil fills in thin lifts or layers to form embankments. In some cases, compaction is used to refill sections where unsuitable soil--such as peat or shallow, soft clay--has been removed. Compaction is also used to treat shallow, loose soils by excavating them from an area and replacing them in the same place and compacting them in layers or lifts. In this manner, a denser, stronger subgrade is provided. Finally, compaction is used to densify loose soils in-place to a limited depth.

Description - Choosing the correct compaction method depends on the soil characteristics described below.

a. Cohesionless soils comprise sands, gravels, and mixtures of the two. These are included in groups classified by USC codes GW, GP, SW, and SP. Granular soils with a significant fines content (i.e., more than 12 percent finer than a No. 200 sieve) are described in the following subsection, although the methods presented here may also be used successfully.

Although almost any type of equipment can be used to compact granular soils, the most effective method by far is by applying vibratory loads. Limited vibration can be achieved by tamping or by dropping weights onto the surface of the soil. However, vibratory compaction is greatly enhanced if the compactor operates near the resonant frequency of the compactor-soil system. There are two forms of vibratory compactors. Vibratory plates, generally from 0.3m to 1m (12 inches to 36 inches) square, operate at a frequency of about 1,000 to 4,000 vibrations per minute; smaller plates operate at higher frequencies. Plate compactors are used most frequently for work in confined areas--such as in trenches or adjacent to structures--and are suitable for compacting thin lifts. Vibratory rollers are used in larger unconfined areas and when large-volume cohesionless fill compaction is required. Rollers may be as small as 0.5m (20 inches) wide and weigh 220 kg (500 pounds) or as heavy as 20,000 kg (44,000 pounds) and with drum widths that are more than 2.5m (8 feet) wide. The dynamic force of the drum is generally 150 percent to 200 percent of the static drum weight.

The maximum lift thickness and minimum number of passes required to compact subgrade soil depends on the size of the equipment used. Recommended limitations on lift thickness and number of passes in the use of vibratory equipment are shown in Table 5-5. Before using thicker lifts, field tests should be planned, as described in the following material under "Engineering", to determine that adequate compaction is achieved. If thicker lifts are used, segregation of coarse particles from the finer soil fraction must be avoided. The number of passes at each location should not be reduced without careful consideration, because this will decrease the potential for producing a uniform fill density. The normal operating frequency of vibratory compaction equipment should be close to the resonant frequency of the compactor-soil system for average soil conditions. However, the performance of vibratory equipment can be enhanced by performing field tests with the equipment operating at a range of frequencies to determine the resonant frequency for specific conditions.

The water content of cohesionless soils generally is not critical for successful compaction. The reasons for this are that the soil has a high internal frictional strength and that high permeability permits rapid drainage of water from saturated soil. However, bringing the soil to near optimum water content will aid compaction. In fact, one type of cohesionless soil can only be satisfactorily compacted if it is near its optimum water content. Clean, fine to medium sands have no inherent unconfined shear strength. Unless there is sufficient water to provide capillary tension, the vibratory action loosens the soil as rapidly as it is densified. Fine to medium sands must be kept moist after compaction to prevent disturbance by equipment operating on the compacted surface. Saturation removes capillarity, and a vibratory compactor will often sink into saturated, fine to medium sand.

Large vibratory compactors are used to densify cohesionless soils in place. The effective depth of treatment depends on the size of the equipment, the gradation of the soil, the in-place water content, and the required degree of compaction. Many passes of the equipment, up to 15 or 20, may be beneficial.

TABLE 5-5. COMPACTION LIMITATIONS FOR COHESIONLESS SOILS

Compaction Method	Maximum Stone Size	Maximum Loose Lift Thickness: Within 4 feet of Bottom of Ties	Maximum Loose Lift Thickness: Less Critical Areas	Minimum Number of Passes: Within 4 feet of Bottom of Ties	Minimum Number of Passes: Less Critical Areas
Hand-operated vibratory plate or light roller in confined areas	4"	6"	8"	4	4
Hand-operated vibratory drum rollers weighing at least 1000# in confined areas	6"	10"	12"	4	4
Loaded 10-wheel truck or D-8 crawler	6"	10"	12"	4	2
Light vibratory drum roller minimum weight at drum 8,000#; minimum dynamic force 10,000#	8"	12"	12"	4	2
Medium vibratory drum roller minimum weight at drum 10,000#; minimum dynamic force 20,000#	8"	18"	18"	6	4

The depth of treatment will be at least the maximum lift thickness shown in Table 5-5, and the maximum depth may extend from 1.3m to 2.5m (3 feet to 8 feet), depending on conditions and the density results to be achieved. Well-graded materials (i.e., having a wide range of particle sizes) can be more easily compacted than can uniform size soils. Saturating in-place, cohesionless soils with broad gradation prior to applying vibration will aid in-place treatment because (1) surface tension will be removed, thereby permitting easier rearrangement of particles; and (2) the seepage pressures of the water will contribute to the compaction. Cohesionless soils can be densified by water alone by using methods called puddling or jetting; however, these methods do not achieve high densities, and the results are sometimes erratic.

b. Moderately cohesive soils include silty and clayey sands and gravels. (These soils are classified as GM, GC, SM, and SC.) Clayey sands and gravels with more than about 30 percent to 45 percent fines may behave more like the plastic soils described below. The effectiveness of vibratory compactors decreases with increasing cohesion. Densification of cohesive soil is best accomplished by applying high shear stresses to induce rearrangement of the soil particles and by using high pressures to squeeze out the air-filled voids. Because the permeability of these materials is low, it is not possible to compress completely saturated materials.

Two types of rollers have been found to effectively compact these soils. Pneumatic-tire rollers with several separated tires per axle are best suited for compacting low-plasticity, silty soils. The tire inflation pressure (which is equal to the soil contact pressure) generally is 350 kPa to 860 kPa (50 psi to 125 psi). Lift thickness is limited to 150mm to 300mm (6 inches to 12 inches) for 25-ton to 50-ton rollers; lifts of 300mm to 450mm (12 inches to 18 inches) may be compacted by using 100-ton rollers. Four to six passes usually are satisfactory to achieve compaction.

A sheepsfoot roller is covered by feet spaced 200mm to 300mm (8 inches to 12 inches) apart, which are 75mm to 200mm (3 inches to 8 inches) across, and which generally protrude at least 230mm (9 inches) from the drum. These feet produce a contact pressure of 2068.2 kPa to 4136.4 kPa (300 psi to 600 psi), and vibratory impact drivers are sometimes added to increase the pressure. Large shearing strains are developed in the soil by the feet, and high contact pressures reduce the voids. The maximum effective lift thickness is 150mm to 300mm (6 inches to 12 inches) to permit the feet to reach in to compact the soil. Generally, four to six passes will result in suitable densification.

For compaction in confined areas, tampers are usually used; powered tampers apply 500 to 1,000 impacts per minute to produce a high contact stress over a 150mm to 300mm (6-inch to 12-inch) contact plate. Lift thickness should not exceed 150mm (6 inches), and equipment can be manually operated or attached to a backhoe arm.

Regardless of the equipment used, compaction of cohesive soils is critically dependent on the soils' water content. The higher the fines content, the more important is moisture control. The water content must be within 2 percent or 3 percent of the optimum compaction water content. For uniform, nonplastic silts, the placement water content must be controlled within 1 percent of the optimum, or they cannot be compacted at all. Compaction control is discussed in greater detail in the section on Engineering, which follows the next subsection.

c. Highly plastic soils generally refers to clay (USC classes CL and CH). When excavated, clay often comes out in chunks. When placed in the fill, compaction breaks down the chunks, so that open voids between them are eliminated. A heavy sheepsfoot roller is effective in accomplishing this. Lift thickness should not exceed 150mm to 200mm (6 inches to 8 inches). The results are best achieved when the clay is placed at a moisture content slightly above the plastic limit. If drier, the chunks are too hard to break down, and if much wetter, the clay will be sticky and will either clog the roller or cause the equipment to lose traction. In confined areas, tampers are often used with a lift thickness of 100mm to 150mm (4 inches to 6 inches).

Engineering - The standard exploration and classification procedures discussed in Secton 2.1 can be used to determine the need to compact subgrade soils. Standard penetration tests (SPT) are frequently used to determine in-situ compactness of granular soils. Cohesive soil strength also can be roughly gauged using SPT, cone tests, or vane shear tests as indicated in Section A5.9 and A5.5.

To evaluate methods of compaction, soil classification and grain size characteristics must be known. Use of the various Unified soil groups in compacted fill are indicated in Columns 9 and 10 of Table A-1.1. Typical engineering properties of compacted soils are shown in Table A-1.2. Sieve analyses should be performed on samples of potential compacted subgrade fill. If more than 10 percent to 15 percent of the material is finer than the No. 200 sieve, Atterberg limits of the minus No. 40 sieve fraction should be determined to evaluate a soil's plasticity.

Where granular materials are economically available, they should be used rather than cohesive soils because (1) they can be compacted with less effort; (2) moisture content control is less critical; and (3) the resulting fill is stronger and less compressible. For general compacted fill from off-site borrow pits in the northeastern United States, fines content should be limited to no more than 30 percent and the plasticity index (liquid limit minus plastic limit) to no more than 15 percent. AREA Manual, Section 1.3.5.13 recommends a 12 percent limit on the plasticity index.

In other areas, such as the southeastern or central United States, where gravel is scarce, sand and clay materials can be used with a higher fines content and greater plasticity. In such cases, swelling clays must be excluded; Table 2-4 indicates limits for acceptable materials. Other than organic, highly plastic, or swelling soils, nearly any earth material can be incorporated in a compacted fill, provided that (1) sufficient care is given to moisture control and compaction, (2) proper drainage is achieved, and (3) the track-induced stresses are kept to a level commensurate with the strength achieved in the compacted fill.

Measuring the final in-place dry density of earth fill is the standard and, generally, best means to determine that the compacted fill material is satisfactory. Merely observing that a specified procedure is followed does not guarantee that the end product will be satisfactory. Measurement of in-place total density can be achieved by any of the following procedures:

a. Sand Cone Method (ASTM D1556)--suitable for a wide range of soil types.

b. Rubber Balloon Method (ASTM D2167)--suitable for materials with limited percentage of coarse, angular particles.

c. Nuclear Method (ASTM D2922)--suitable for a wide range of soil types; should be correlated with results by method "a" or "b" on a site-specific basis; difficult to use if material gradation and plasticity vary from place to place and for very coarse materials.

The problems associated with measuring in-place density of materials with large open voids were presented in Section 3.2 concerning measurement of ballast density. The same limitations are associated with measuring in-place density of any fill material containing open voids. It is practically impossible to measure the density of large-size rockfill; satisfactory placement of rockfill is typically determined by observing the method of compaction.

The procedures listed above all determine in-place <u>total</u> unit weight. The strength and stiffness of the fill is determined by the in-place dry unit weight--i.e., the packing of the solid particles. To convert from total to dry unit weight, the water content of the soil is determined after weighing and drying it (ASTM D2216 or C566) or by applying nuclear methods (ASTM D3017).

To evaluate the suitability of the measured in-place dry unit weight, a reference unit weight must be determined. Most often the Proctor compaction test is used (see Section 4.2 on "Subballast"). As discussed in 1967 by Terzaghi and Peck in <u>Soil Mechanics in Engineering Practice</u>, the maximum density achieved in the field and by the compaction test depends on the placement water content and the amount of compaction or energy applied. The water content at which the maximum dry unit weight is achieved--i.e, the optimum water content--decreases with increasing compactive effort. It is important to apply a compactive effort in the laboratory that corresponds to the energy imparted by the compaction equipment in the field, so that the laboratory-derived optimum water content corresponds to the field compaction situation.

For light equipment, the Standard Proctor test is suggested (ASTM D698); the Modified Proctor test (ASTM D1557) is used with modern, heavy equipment. A field test section can be constructed by placing fill of varying water contents, using the actual procedure to be adopted during construction, and measuring the resulting in-place dry density to develop a relationship between the two. For soils where placement water must be closely controlled, such as nonplastic silts, selecting an appropriate reference density test is important.

Criteria for a satisfactory minimum density of fill are recommended in Section 1.3.5.15 of the AREA Manual. Earth fill in embankment should attain at least 95 percent of the maximum dry density as determined in the Standard Proctor test. Although this recommendation is reasonable, it may be advantageous to require a higher density, such as 98 percent to 100 percent of the Standard Proctor maximum dry density for the 0.3m to 1.3m (1 foot to 4 feet) of fill immediately below the ballast and subballast. The Modified Proctor density (ASTM D1557) may also be used as a reference for minimum density criteria. In general, there is about a 5-percent density difference in the tests--i.e., 95 percent of the Standard Proctor maximum corresponds to 90 percent of the Modified Proctor Density. However, because the two tests will determine different optimum water contents, the test used should correspond with the type of equipment being employed so that the optimum water content applies to the field conditions.

Cost - Compaction costs are affected by the type of material, access to the area, and the compacted density required. Large rollers are more economical than smaller tampers. However, the mobilization (startup) cost is higher for the big equipment, and it can only be used in an open area. Compaction by rollers in 300mm (12-inch) lifts costs about $1.25 to $2.00 per cubic yard. If the fill is highly plastic clay or nonplastic silt, the cost may be 50 percent higher than listed. The time of year, expected weather, and water availability also directly affect the cost.

Deep densification of subgrade using a roller compactor operating from the surface will cost about $500 to $750 per day. The cost per square yard depends on the roller width, operating speed, and the number of passes required to accomplish compaction. The typical cost is $0.05 to $0.10 per square yard for 10 passes over an area, with the roller compactor operating at 5 miles per hour.

Discussion - When subgrade conditions are suitable, compaction probably is the most economical and most frequently used means of soil stabilization. Surface proofrolling of exposed subgrades should be considered in all cases (for both cuts and fills) prior to placing an embankment on the track substructure. One exception to this is when the subgrade might be disturbed by compaction, such as when there is saturated clay or silt close to the water table.

Compaction has limited value for stabilizing existing track subgrades. The effective depth of compaction from the surface is only 3 feet to 8 feet, depending on the subgrade material, groundwater location, and other conditions. The compaction equipment may not be as effective as an operating train. If track reconstruction is required, surface rolling should be considered after removal of the superstructure and sledding of unsuitable ballast--provided subgrade conditions would not be adversely affected.

Admixtures

Consider a natural subgrade that cannot accept the applied stresses of the track and its operating loads or does not remain stable in track. Some soils, such as saturated clays or uniformly graded sands, cannot be adequately treated by compaction in-place or by excavation/recompaction described previously. In this situation, often the most economical method of treatment is to excavate and replace the natural soil with a suitable borrow material. However, if acceptable replacement materials are not economically available, treating the natural soil with admixtures or stabilizers that facilitate compaction and produce a stable subgrade may be the best alternative. Even if the subgrade is adequate, it may be economical to improve the subgrade with a stabilizer to increase service life; i.e., decrease maintenance frequency.

Five admixtures or stabilizers are commonly used to stabilize soil, particularly for highway subbase and subgrade construction. These are Portland cement, lime, bitumen, fly ash, and clays. Other stabilizers are also used, but infrequently, because of the high cost of the admixture. Some of these will be mentioned briefly. All of the admixtures require certain common elements for their use, as discussed below.

Using a soil admixture, stabilizer, or modifier requires the following general steps: application or spreading, mixing, moisture control, pre-curing (sometimes), compaction, and curing. This sequence implies disturbance of the subgrade soil, so that these methods generally are applicable only to new construction or reconstruction of the track substructure. Methods that use some of these same materials yet that do not require disruption of the track are discussed in Section 5.2 under "In-place Modification."

Most of the admixture materials, other than bitumen, are manufactured in powder form and are supplied in bags or in bulk. Bagged materials often are spread manually with some assist from bulldozers or other equipment. Bulk powder carriers may be fitted with spreader bars to permit direct spreading of materials from the delivery truck. In some cases, the active stabilizer is mixed with an inert powder, such as ground rock flour, to increase the volume of the additive to promote distribution of the admixture through the soil. Sometimes admixtures can be spread as liquid suspensions if the soil is not too wet for compaction. Bitumen is always applied in liquid form.

Mixing is best accomplished by special mixing/pulverizing equipment that uses rotating tynes to break up and mix the soil and stabilizer. Some of this special equipment excavates a layer of soil, transports it to a mixing drum, and deposits the modified soil out the rear end of the drum in a continuous moving operation. In some cases, particularly if bituminous stabilizers are used, the soil is excavated, transported to a central batch plant, and treated before being returned to the site. Nonspecialized equipment is also frequently used; this includes disc or tyne harrows, bulldozers, ripping bars, motor graders, and scrapers. Although these types of equipment are less efficient, they generally cost less to mobilize than special mixers and are appropriate for small projects.

Moisture control refers to controlling water content to achieve an adequate compacted density. An admixture, such as lime, can be used to permit placement of soil at its natural water content. Drying can be accomplished by exposing the loose soil to the sun and air, providing the weather cooperates. Watering can be accomplished by watering from trucks over the surface before mixing or by using spray bars that are part of the special mixing equipment. The admixture, in particular lime, is sometimes supplied in a slurry form so that the admixture and water are added together. Where applicable, this aids in the mixing because the slurry is of a larger volume and is easier to disperse throughout the soil.

Pre-curing refers to allowing the treated soil to stand prior to compacting it. This is often desirable with bitumen stabilization. A relatively high volume of volatile solvent will aid in the mixing process; however, compaction is sometimes achieved with less solvent. In lime stabilization, higher compacted strength is achieved if the chemical reaction partially proceeds before compaction. Pre-curing accomplishes this.

Compaction is carried out using the procedures previously described. In evaluating the type of equipment to be used, the modified soil properties should be considered. Sheepsfoot rollers have the advantage of promoting additional mixing and pulverization during the compaction process.

Final curing causes the admixture to cement and achieve adequate strength prior to applying load. In some cases, if the undercured, stabilized soil is overloaded, the cemented bonds will be broken and final strength will be impaired--although some of the stabilizers continue to gain strength over the long term and are "self-healing."

Cement

Application - Portland cement may be used to stabilize almost any soil except highly organic, high salt, or high sulphate materials; these components interfere with the Portland cement reactions. Coarse gravels (GP) greater than 3/4 inches in size are not suited for cement stabilization because of their high cement content requirement, and heavy clays (CH) create mixing difficulties. Widely-graded granular materials (G and S classes) that possess a floating aggregate matrix--i.e., the fine fraction of the soil is inadequate to hold the coarse particles in place--are best suited for cement stabilization(1).

Generally cement stabilization should be limited to soils with less than 35 percent passing the No. 200 sieve and a plasticity index of less than about 30 percent. Materials exceeding these percentages probably can be more efficiently stabilized with lime. In some cases, lime and cement can be combined to treat highly plastic material as described in the following section on lime.

Description - Cement stabilization works by producing a very lean concrete called soil cement. The cementing action is produced by a hydration reaction within the Portland cement components themselves. The hardened cement adds

---

(1)J.A. Epps et al., "Soil Stabilization: A Mission Oriented Approach," Highway Research Record, No. 351, p. 4.

cohesion by bonding together soil particles and soil aggregations. The principal difficulty in working with this method is that the cement begins to set in about two hours after wetting. Mixing and curing moistened soil cement must be completed before setting begins because disturbance after initial setting will decrease the final strength. If the in-place soil is dry, setting will be delayed until after water is applied to the soil-cement mixture. Cement stablization achieves the following subgrade performance improvements:

a. It increases cohesion, unconfined compression, and tensile strength.

b. It increases stiffness (modulus).

c. It increases resistance to wet-dry and freeze-thaw exposure.

d. It lowers permeability (yet increases it for clays).

e. It decreases swelling in plastic clays.

Engineering - In addition to the conventional exploration, testing, and engineering evaluation required for all track substructure engineering (described in Section 2.1), cement stabilization requires evaluation of the required quantity of cement and compaction characteristics for the stabilized soil. Preliminary evaluation of cement requirements can be based on information in Table 5-6. However, final determination of cement requirements should be based on a program of laboratory testing. In Great Britain, soil cement design for highways is based on selecting a cement content that yields a minimum unconfined compressive strength of 1.7 MPa (250 psi) for compacted and cured specimens. In the United States, freeze-thaw and wet-dry exposure durability (ASTM Method D559) are used as design criteria(1). Losses of 7 percent to 14 percent are the typical allowable limits after 12 environmental exposure cycles. Sandy and gravelly soils generally experience greater exposure losses than finer-grained soils, yet the coarse-grained soils usually achieve higher compressive strengths.

The compressive strength of soil cement produced in the field usually is about 60 percent of the strength of material manufactured in a laboratory. For this reason, 1 percent or 2 percent of extra cement is added in the field. However, it has been observed that cracking of soil cement increases with greater cement content; this has produced difficulties in highways because the cracks extend through the final pavement. Cracking is not expected to directly affect the performance of ballasted track; however, cracks increase the permeability of the soil cement layer, which may lead to indirect degradation due to the saturation of the underlying, untreated soil.

---

(1)O.G. Ingles and J.B. Metcalf, Soil Stabilization, John Wiley & Sons, New York, 1973, p. 120.

TABLE 5-6. CEMENT REQUIREMENTS FOR VARIOUS SOILS

AASHO Soil Classification	Unified Soil Classification[a]	Usual Range in Cement Requirement[b] Percent by Volume	Usual Range in Cement Requirement[b] Percent by Weight	Estimated Cement Content Used in Moisture-Density Test (percent by weight)	Cement Content for Wet-Dry and Freeze-Thaw Tests (percent by weight)
A-1-a	GW, GP, GM, SW, SP, SM	5 to 7	3 to 5	5	3 to 5 to 7
A-1-b	GM, GP, SM, SP	7 to 9	5 to 8	6	4 to 6 to 8
A-2	GM, GC, SM, SC	7 to 10	5 to 9	7	5 to 7 to 9
A-3	SP	8 to 12	7 to 11	9	7 to 9 to 11
A-4	CL, ML	8 to 12	7 to 12	10	8 to 10 to 12
A-5	ML, MH, OH	8 to 12	8 to 13	10	8 to 10 to 12
A-6	CL, CH	10 to 14	9 to 15	12	10 to 12 to 14
A-7	OH, MH, CH	10 to 14	10 to 16	13	11 to 13 to 15

[a]Based on U S Air Force recommendations (2)
[b]For most A horizon soils, the cement content should be increased 4 percentage points if the soil is dark gray to gray and 6 percentage points if the soil is black

Reproduced from "Soil Stabilization: A Mission-Oriented Approach," p. 10, by J. A. Epps et al. Year of first publication: 1971.

Cost - The cost of cement stabilization is comprised of purchasing and delivery of cement, spreading, mixing, compacting, and laboratory testing. Cement delivered in bulk will cost about $50 to $70 per ton (907 kg), and bagged cement costs about 50 percent more. Spreading, mixing, and compacting the soil cement costs about twice that of simple compaction, or about $2.50 to $4.00 per cubic yard. For heavy clay soils, the cost would be about 50 percent higher. The cost of a laboratory testing program that includes a series of compaction and unconfined compression or wet-dry cycle tests is estimated to be between $1,000 and $2,000.

Based on these data, cement stabilization costs can be calculated for a typically treated railroad track subgrade. Assume the soil will be treated to a depth of 300mm (12 inches) and a width of 4.5m (15 feet). Also assume the soil has an untreated dry unit weight of 1.6 Mg/cu. m) (100 pcf) and 8 percent cement is required. The estimated cement stabilization treatment cost is $4.00 to $7.00 per track foot, not including engineering and laboratory testing costs.

Discussion - The principal advantage of Portland cement stabilization is to permit use of local materials of secondary quality in subgrades when replacement by suitable offsite borrow is too costly. The main difficulty is the short time within which to compact after the cement mixes with water. Cement improves mechanical, permeability, and environmental performance characteristics and generally limits surface water infiltration into the subgrade if cracking is not severe. It is particularly effective in stabilizing poorly graded sands; a small percentage of cement will impart a high unconfined strength to a material that otherwise would be an unstable, shallow subgrade that might foul the ballast. Experience using soil cement in railroad track is limited. Its successful use is reported on the Southern Pacific, both alone and in combination with filter fabric(1). The major uncertainty is the durability of the cement under track loads. If the soil cement is overloaded, the cement bonds will be broken, and recementing will not occur. Further study of soil cement used as a subgrade or subballast and its long-term permeability and durability in track is needed.

Lime

Application - Lime may be used to treat most soils with at least some clay mineral fraction, i.e., clays, silts, and silty or clayey sands and gravels (USC codes CH, CL, MH, ML, SC, GC). Clay mineral content (finer than 2 microns) should be at least 7 percent by weight, and the plasticity index should exceed 10 percent. Lime stabilization is used to decrease plasticity, improve workability, reduce swelling potential, and increase strength. Sometimes, lime is used to improve workability so that some other additive, such as cement or bitumen, can be mixed and compacted; however, certain soils do not react with lime and therefore cannot be treated with it alone. Laboratory testing, as described under Engineering, is required to determine if a soil is lime reactive.

Description - Soil stabilization with lime refers to the use of either of two chemicals, calcium oxide (CaO), i.e., quicklime, or calcium hydroxide ($CaOH$), called hydrated or slaked lime. These are produced from calcium carbonate ($CaCO_3$) or limestone by heating in a kiln. Limestone itself is not used as a soil stabilizer except infrequently as an extender or filler to promote the mixing of active stabilizers.

Lime stabilization works by chemically combining the lime with the clay minerals and soil moisture to form calcium silicates, the same bonding compounds formed by Portland cement. The clay minerals are a necessary component in the reaction. As they react, plasticity decreases immediately. This effect is dramatic for montmorillonite clays and minor in kaolins. Quicklime has another effect: the calcium oxide combines with water to form hydrated lime, releasing oxygen and causing excess water to be absorbed from the soil.

---

(1)"Engineering Fabrics - Used and Researched by the Southern Pacific," Progressive Railroading, Vol. 22, No. 3, March 1979, p. 56.

However, quicklime is hygroscopic--i.e., it has a strong affinity for water. This makes storage a problem because it tends to combine with moisture in the air. The hydrating reaction also releases substantial heat. This makes quicklime a hazardous substance for workmen to handle.

Lime also alters compaction characteristics. The maximum dry density of the treated compacted soil is less than, and the optimum water content is greater than, the untreated soil. The lower in-place density is more than compensated for by the cementing action, resulting in a higher shear strength. Because clayey soils often are wetter than the optimum moisture content in their natural state, increasing the optimum water content aids compaction. Unlike Portland cement, the cementing reactions of lime take place slowly, so that working time prior to compaction is flexible. Pre-curing often results in higher final strength.

Lime is sometimes used in combination with other agents. With Portland cement or bitumen, lime is used to improve workability of the soil and promote mixing and compaction. Fly ash or other pozzolans are sometimes combined with it to provide the clay mineral component required for reactivity in clean granular soils.

Engineering - Besides the conventional engineering practices associated with track substructure, there must be an evaluation of the correct quantity of lime to use to achieve the desired results. Because lime reacts with the clay minerals in soils, the quantity of lime is directly related to clay content. A range of 3 percent to 7 percent by weight lime will treat most soils. Ingles and Metcalf, in their 1973 book, Soil Stabilization, indicated that about 1 percent by dry weight of lime is required for each 10 percent by weight of clay (finer than 2 microns). The most rapid way to determine an approximate required lime content is by using the pH test developed by Eades and Grim. The pH (acid-base index) of lime and water solution is 12.4 (a strong base). First, soil of a known weight is mixed with water. Lime is then added progressively until the measured pH rises to 12.4 after one hour of mixing, indicating that all clay minerals within the soil have reacted with the lime. Final selection of required lime content should be based on a program of Proctor compaction tests and laboratory unconfined compression tests on compacted samples with lime content ranging within ±2 percent of the quantity determined by the pH test.

The desired unconfined compression strength of soil-lime mixtures depends on mechanical and environmental exposure. Definite criteria have not been developed for railroad engineering. However, criteria do exist for pavement practice. Table 5-7 indicates the required unconfined strength for lime-soil mixtures placed at different depths below pavement and exposed to various numbers of freeze-thaw cycles during the first service season. Only the first season is considered because continued strength gain will take place during the later warm seasons. This chart may be used as a basis for a railroad substructure design system for lime-soil, in which the residual strength requirement is based on an analysis of the stresses in the track substructure. Thus, the 20 psi requirement may be appropriate for a modified track subgrade beneath 600mm (24 inches) of ballast and subballast. For a lime-stabilized subballast, a value of 30 psi may be appropriate beneath 300mm (12 inches) of top ballast, assuming stress distribution based on a 100-ton car loading.

However, such a design procedure for railroads must be confirmed by trial and in-service testing.

Cost - The cost of the lime-soil stabilization is comprised of the same elements as soil cement. The major differences are related to the quantity and cost of the lime versus cement and the greater workability of lime-treated cohesive soils. Bulk lime costs about $70 per ton (907 kg); bag lime costs about 50 percent more. Spreading, mixing, and compacting are estimated to cost $2.00 to $3.50 per cubic yard. Using the same parameters as in the soil cement example previously discussed--except using 5 percent by dry weight lime rather than the cement--the cost of stabilizing track subgrade to a depth of 300mm (12 inches) is $4.00 to $7.00 per track foot. The cost figure for cement and lime per track foot are not comparable. A clayey soil would probably require 10 percent to 12 percent by weight cement, and the comparable cost would be $6.00 to $8.50 per track foot for stabilization of clay with cement, providing that the cement would produce satisfactory workability to permit the treatment. For granular soils, the cost of cement stabilization typically would be less than lime.

TABLE 5-7. TENTATIVE LIME-SOIL MIXTURE COMPRESSIVE STRENGTH REQUIREMENTS

Anticipated Use	Residual Strength Requirement[b] (psi)	Strength Requirements for Various Anticipated Service Conditions[a]			
		8-Day Extended Soaking (psi)	Cyclic Freeze-Thaw[c] (psi)		
			3 Cycles	7 Cycles	10 Cycles
Modified subgrade	20	50	50	90 50	120
Subbase					
Rigid pavement	20	50	50	90 50	120
Flexible pavement					
10-in. cover	30	60	60	100 60	130
8-in. cover	40	70	70	110 75	140
5-in. cover	60	90	90	130 100	160
Base	100	130	130	170 150	200

[a]Strength required at termination of filed curing (following construction) to provide adequate residual strength.
[b]Minimum anticipated strength following first winter exposure.
[c]Number of freeze-thaw cycles expected in the lime-soil layer during the first winter of service.

Reproduced from "Soil Stabilization: A Mission-Oriented Approach," p. 8 by J.A. Epps et al. Year of first publication: 1971.

Discussion - The principal advantage of using lime is that it produces immediate and dramatic improvements in the workability of clayey soils. Often, lime provides the only way to work with soft, wet clayey soil other than excavating the soil and replacing it. Also, the increase in strength from lime is gradual, so that timing between mixing and compaction is not as critical as with cement. However, if a rapid increase in strength is required, lime will not suffice. On the other hand, lime-treated soils are not as susceptible to cracking as soil cement.

Lime is effective in treating soils with clay. Generally, the permeability of clayey soils will increase with lime. If granular soils require stabilization, it sometimes is economical to add a mixture of lime and fly ash or lime and clay, as discussed further on in this section.

Bitumen

Application - Bitumen is used to cement soils that are mostly granular with limited plastic fines. Properties of soils suitable for bituminous stabilization are listed in Table 5-8. In general, only soils classified under USC codes GW, GP, SW, SP, GM, and SM are candidates. Wet soils usually are poor candidates, because adding the liquid bitumen stabilizer worsens characteristics of wet soils needed for compaction.

TABLE 5-8. ENGINEERING PROPERTIES OF MATERIALS SUITABLE FOR BITUMINOUS STABILIZATION

Item	Sand-Bitumen	Soil-Bitumen		Sand-Gravel-Bitumen
Gradation (percent passing)				
1½-in. sieve				100
1-in. sieve	100			
¾-in. sieve				60 to 100
No. 4 sieve	50 to 100		50 to 100	35 to 100
No. 10 sieve	40 to 100			
No. 40 sieve			35 to 100	13 to 50
No. 100 sieve				8 to 35
No. 200 sieve	5 to 12	Good	3 to 20	
		Fair	0 to 3 and 20 to 30	0 to 12
		Poor	>30	
Liquid limit		Good	<20	
		Fair	20 to 30	
		Poor	30 to 40	
		Unusable	>40	
Plasticity index	<10	Good	<5	
		Fair	5 to 9	
		Poor	9 to 15	<10
		Unusable	>12 to 15	

Reproduced from "Soil Stabilization: A Mission-Oriented Approach," p. 13, by J. A. Epps et al. Year of first publication: 1971.

Description - Soil-bitumen mixtures are made with asphalt cutback, emulsions, or foams. Cutback is asphalt dissolved in a volatile oil, such as kerosene or fuel oil, in about a 50-50 ratio. Emulsions are mixtures of about 55 percent to 60 percent asphalt and water with an emulsifier to promote suspension of the asphalt. Foams are produced by bubbling steam through hot asphalt. All these processes are used to promote mixing of the asphalt with the soil. The heaviest asphalt product that can be adequately mixed with the soil should be used. Bitumen stabilizes soil by cementing particles and soil aggregations together to form a cohesive mass. The solvent or water in asphalt products either must evaporate or be absorbed into the soil before the bitumen will cement.

Bitumen increases soil strength and decreases its permeability. However, if too much bitumen is added, strength will decrease as the bitumen layers between soil particles thicken. To achieve waterproofing, bitumen can be sprayed on the surface of a compacted subgrade and penetrate downward. The thin bitumen surface film limits infiltration of water. Used on "oiled" gravel roads for many years, this process may be effective for railroad track to treat soils that soften in the presence of water--provided that the groundwater level is low enough so that softening is not caused by trapped evaporation. This has been discussed in Section 5.3.

Engineering - The standard engineering explorations, testing, and evaluations described in Sections 2.1 and 2.6 are all required to evaluate bitumen stabilization. Selection of the quantity of bitumen and the most effective method to use is based on measurements of the strength of the compacted specimens. Unconfined compression test (ASTM D1633), California bearing ratio (ASTM D1883), and the Hubbard-Field extrusion test (ASTM D915) have been used. Permeability of compacted bitumen-soil mixtures may be measured by the permeability tests shown in Figures 2-6 and 2-7 (ASTM D2434). Generally, 4 percent to 8 percent by weight of asphalt is required for adequate stabilization.

When evaluating compaction characteristics of bitumen-stabilized soil, the solvent and water in cutbacks and emulsions will affect compaction. Therefore, they should be combined with the moisture content when plotting compaction curves. The addition of bitumen will result in a lower maximum dry density but in increased shear strength.

Cost - Bitumen stabilization is costly and increases with the price of petroleum. The cost is about $0.65 to $0.85 per gallon for cutback and $0.55 to $0.75 per gallon for emulsion asphalt; both contain roughly 50 percent asphalt. Cost factors are the same as for cement and lime. The costs of applying the bitumen and compacting it are somewhat greater because the material is more difficult to mix. Frequently, the mix is produced in a batch plant rather than in-situ. The cost to treat a 15-foot-wide strip that is 1 foot deep was developed assuming that placement costs are about 30 percent higher than for cement (6 percent asphalt was assumed). The treatment cost is about $15 to $20 per track foot, which is about two to three times the cost of using cement or lime.

Discussion - Although bitumen stabilization is expensive, asphalt is readily available. It is attractive for road subgrade stabilization where the equipment used to place paving layers can be used to stabilize the subgrade. Asphalt has been used on railroad track to treat the ballast surface. The primary purpose has been to seal the ballast bed to exclude blown-in sand that sometimes contaminates ballast, particularly in desert areas, and it has been sprayed adjacent to a track to limit the blowing of unstable sandy soils. Asphalt-treated ballast tends to reduce track noise, but asphalt treatment of ballast makes subsequent maintenance of the track difficult.

Miscellaneous Admixtures

Application - There are two general classes of miscellaneous admixtures: combination products, such as lime-cement or lime-fly ash (LFA), and special admixtures that have proved successful either for stabilizing special soils or for using waste products that are economically available. Some of the most common applications are described.

Description - Fly ash is a finely ground siliceous material (containing silica, SiO_2) that is most frequently produced from coal furnace flue dust or from blast furnaces. It also can be produced by grinding natural volcanic ash or sediments. Fly ash, when used in soil stablization, is often combined with lime. In clean granular soils, the fly ash takes the place of naturally occurring clay with regard to its reaction with lime.

The closest competitor for LFA stabilization is Portland cement, either alone or in combination with lime. LFA may be beneficial in soils where the workability is improved by lime but where there is insufficient reactivity to produce the necessary strength gain. LFA has a longer working time than cement. Fly ash costs about $40 per ton versus $60 for cement, although local price variations may reverse the cost ratio.

Clay sometimes is used as an additive for clean, poorly graded, granular soils to increase their stability and decrease their permeability. It can be used in combination with lime in the same manner as fly ash, and can be applied in a dry, powdered form or as damp, natural clay. However, if the clay is wet, it will be difficult to spread and mix. Natural in-place clay may be mixed with unstable aggregates by spreading the aggregate, blading it to a depth to include both the aggregate and underlying clay, followed by spreading and compaction.

Clays are the most troublesome of railroad subgrades. Avoid highly plastic, swelling materials when adding clay to granular materials. To prevent significant strength loss and susceptibility to pumping, the total fraction of mixed material passing the No. 200 sieve should be less than 30 percent, with 15 percent to 25 percent being most desirable for ease of compaction and for achieving a high compacted density(1).

Salt was previously mentioned in Section 5.2 as being applied to the exposed surface of the track to limit frost heaves. It also is effective in treating montmorillonite clays that tend to swell. The salt is mixed in with a thin layer of clayey gravel to produce a low-permeability barrier that keeps moisture from percolating deeper into the expansive subgrade. This process was used on the Genesee & Wyoming Railroad to protect a clay subgrade, as described in the AREA Bulletin in 1967(2). Salt also may be mixed

---

(1)T. Niskiki, "How to Choose Proper Surface Soils for Roadbeds," Bulletin of the Permanent Way Society of Japan, Vol. 10, No. 2, February 1962, p. 3.
(2)"Salt Stabilization of Subgrade," Bulletin, AREA, Vol. 68, 1967, p. 504-5.

with thicker layers of frost-susceptible soil to decrease ice lens formation. However, in all salt applications, the salt remains soluble and may be leached out over time. Reliable long-term performance observations of salt stabilization are not available.

Discussion - These and other materials have found successful applications for soil stabilization. They are used infrequently because of their high costs and because of their ability to treat only special soils. The design of soil admixtures requires a specialist's knowledge of soil technology, particularly for special stabilizers, and is beyond the scope of this report.

5.5 EMBANKMENT STABILIZATION

Stability failure of embankments was discussed briefly in Section 2.7. Instability occurs when the combined shear strength of the embankment fill, subgrade, and foundation soils cannot support the weight of the embankment and supported structures. To understand methods for improving the stability of an embankment or cut slope, one should have a basic understanding of ways to assess the factor-of-safety against stability failure. The methodology used most frequently to analyze slope stability involves an evaluation of limiting static equilibrium.

The cross section of an embankment situated on a deep soil foundation is depicted in Figure 5-22. Also shown is a potential failure surface that is formed by a circular arc with center at 0 and radius R. Shear displacement on this failure surface will result in a large rotation of the material or sliding mass above the failure surface about point 0. The factor-of-safety may be defined as the sum of all shear strength forces resisting rotation about point 0, divided by the forces causing the rotation. Because rotation is involved, the ratio of moments about point 0 are compared.

As discussed in Section 2.2, the shear strength of soil is often represented by the Mohr-Coulomb failure criterion in terms of effective stress (as displayed in Figure 2-1) and by the equation:

$$\tau_{ff} = c + (\sigma_{ff} - u) \tan \bar{\phi}$$

where

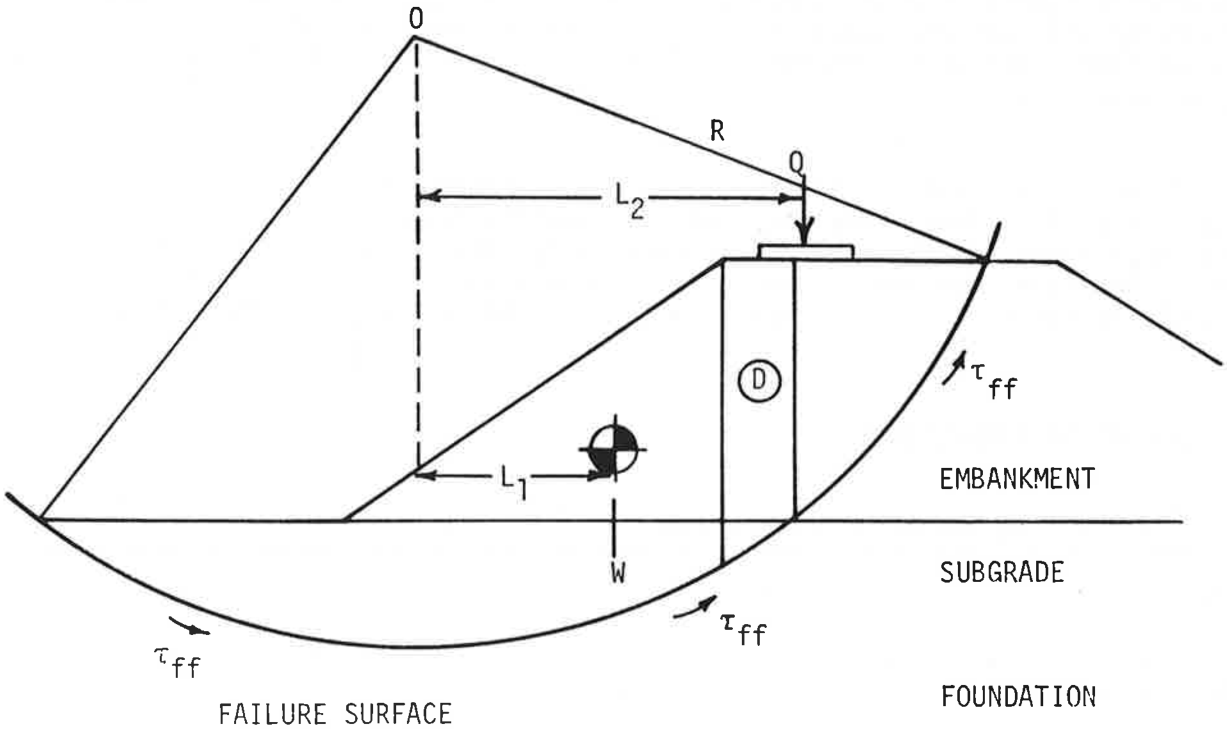
τ_{ff} = shear stress acting on the failure plane at failure; shear strength

$\bar{c}$ = cohesion intercept of the failure envelope

σ_{ff} = total effective normal stress on the failure surface

u = pore pressure

$\bar{\phi}$ = slope of failure envelope, effective friction angle.



$$FS = \frac{\text{SUM OF RESISTING MOMENTS}}{\text{SUM OF DRIVING MOMENTS}} = \frac{\int (\tau_{ff} \cdot dl) \cdot R}{W \cdot L_1 + Q \cdot L_2}$$

FS = FACTOR OF SAFETY

$\int$ = INTEGRAL OR SUM OVER THE LENGTH OF THE ARC

τ_{ff} = SHEAR STRESS ON THE FAILURE SURFACE AT FAILURE

dl = INCREMENTAL ARC LENGTH

R = RADIUS OF FAILURE ARC

W = WEIGHT OF SURCHARGE (TRAIN, TRACK, ETC.)

L_1, L_2 = MOMENT ARMS OF DRIVING FORCES

 = CENTER OF GRAVITY OF SLIDING MASS

Q = SURCHARGE LOAD

FIGURE 5-22. ANALYSIS OF SLOPE STABILITY

To evaluate σ_{ff}, the sliding wedge is broken into a number of slices, such as typical slice D shown in Figure 5-22. By making simplifying assumptions about the forces between slices, the total stress at the bottom of the slice, σ_{ff}, can be calculated, enabling the shear strength, τ_{ff}, to be determined. To evaluate undrained shear failure, the normal practice is to set $\bar{\phi} = 0$ and to set the value of cohesion, $\bar{c}$, equal to the undrained shear strength (S_u). The driving moment inducing rotation is then determined by computing the weight of the sliding mass and multiplying it by the moment arm between the center of gravity and the center of rotation. The factor-of-safety computation is shown in Figure 5-22.

A complete discussion of analyses of embankment failures is beyond the scope of this report. Textbooks, such as Soil Mechanics, by T. W. Lambe and R. V. Whitman, or the comprehensive treatment in 1973 by N. Janbu in Embankment-Dam Engineering, provide complete descriptions of analysis methods. Various methods are used that provide slightly different answers. However, the potential errors in evaluating soil shear strength can be significantly greater than errors due to the analysis methods used. If a slope has already failed, this condition offers the opportunity to verify the methods for evaluating soil properties in a case where the factor-of-safety is known to be equal to 1. From this known condition, the change in factor-of-safety due to stabilization measures can be evaluated with greater confidence.

Before proceeding, it is important to set forth one caveat. Analytical methods that evaluate limiting equilibrium determine an upper bound for the factor-of-safety. To determine the critical factor-of-safety, all potential failure mechanisms and surfaces must first be identified and evaluated. The critical factor-of-safety is the lowest value of all the possible mechanisms (i.e., the lowest of all upper bounds). The significance of this fact is that, when evaluating the causes and cures for unstable slopes, one must look beyond the causes of immediate slope movements. By correcting an immediate problem, it may result in some other type of failure, and slope movements may continue to happen.

With this general understanding of evaluating limiting equilibrium and slope stability, the means for improving stability can be identified--i.e., by increasing the shear strength of the material along the failure surface (or adding resisting force elements), and/or by decreasing the driving moment, which is related to the weight of the fill, as discussed below.

Embankment Widening

Application - If an embankment failure (such as that indicated in Figure 5-22) is taking place or is predicted by analysis before construction, widening the embankment can halt the failure. This can be accomplished by adding berms at the toe of the slope or by flattening the slope angle, as shown in Figure 5-23. Construction of berms is more common and generally easier than changing the angle of the slope. Embankment widening is effective for failures in all soils, although failures are most likely to occur in soft, cohesive soils (e.g., USC classes MH, CL, CH, OL, OH).

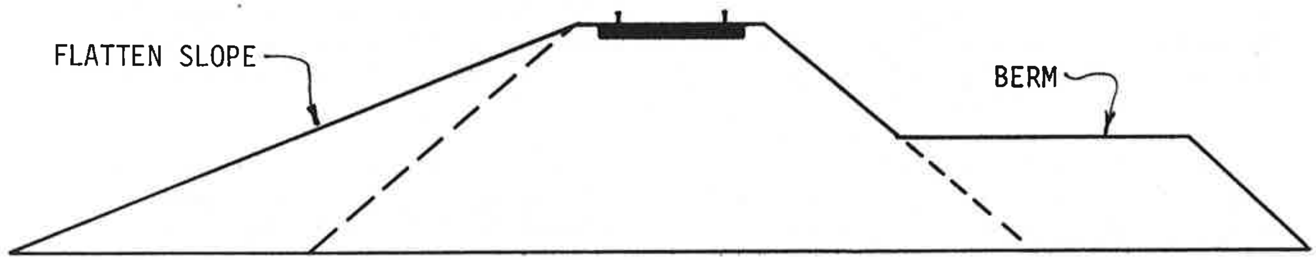


FIGURE 5-23. EMBANKMENT WIDENING

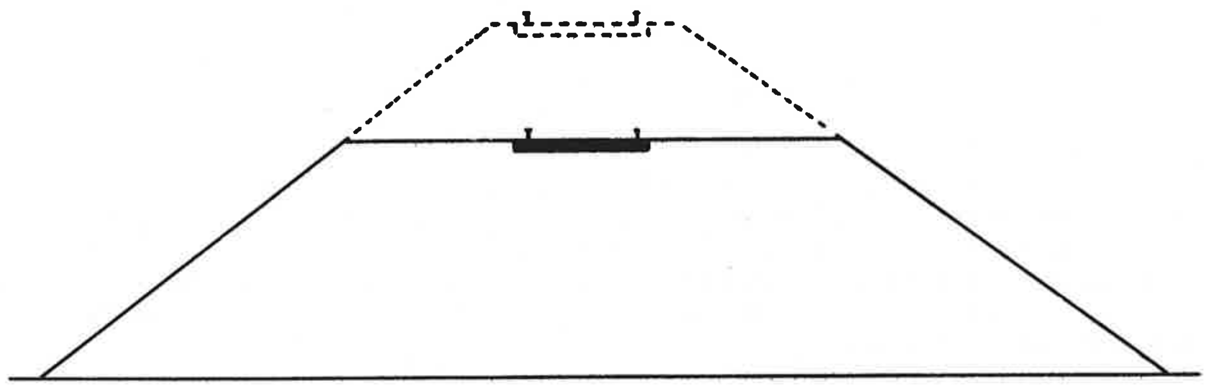


FIGURE 5-24. EMBANKMENT LOWERING

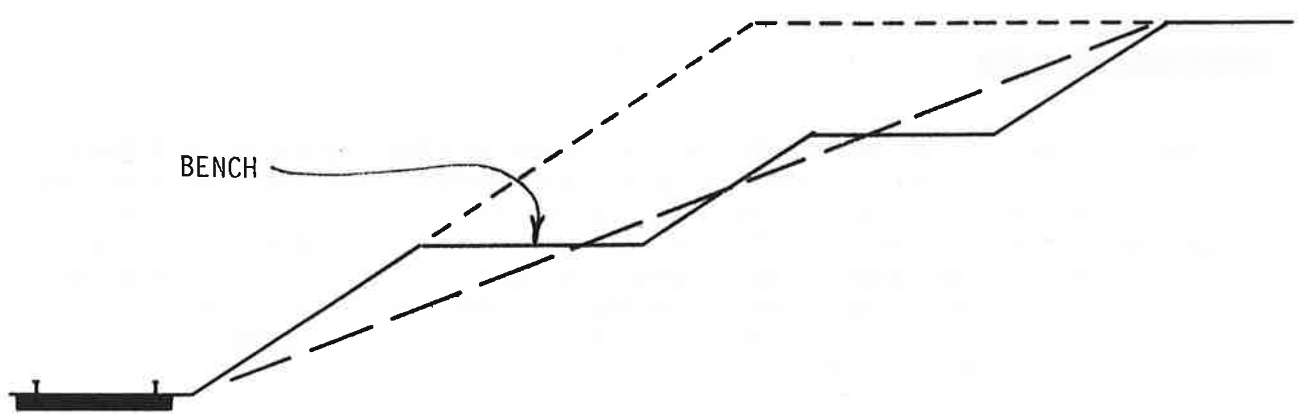


FIGURE 5-25. BENCHING OF CUT SLOPE

Description - Embankment widening improves stability by shifting the center of gravity of the sliding mass closer to the center of rotation, thus reducing the driving moment due to embankment weight. Normally, the shear strength of the added fill does not significantly add to the resisting forces and should be ignored when analyzing the factor-of-safety.

Widening an embankment requires addition of embankment fill. Conventional earthwork equipment, such as trucks, hopper cars, bulldozers, loaders, water trucks, and compactors, is the only equipment needed. The material used can be any locally available common earth borrow; however, granular soils or low-plasticity cohesive soil are preferred for ease of handling and compaction.

Because the strength of the added fill is not of primary importance, the fill may be compacted to a relatively modest in-place dry unit weight, 90 percent of the Standard Proctor maximum (ASTM D698). However, if the berms are greater than 10 feet high or will support structures such as roads or buildings, compaction should achieve a minimum density of 95 to 100 percent of the Standard Proctor maximum dry unit weight. The value selected depends on local practice, types of material available, and design practices for the supported structures.

Engineering - Design of embankment changes should be based upon a thorough engineering study of the existing and projected stability for the modified embankment. This study should include subsurface explorations, soil sampling, laboratory index tests, and engineering property tests. Depth and spacing of explorations are discussed in Section 2.1 of this report.

Where a major construction effort is required, undisturbed samples of cohesive soils should be secured for triaxial unconsolidated-undrained or consolidated-undrained shear strength measurements. Field vane or other in-situ tests aid in evaluating subgrade and foundation strengths of cohesive soils. Embankment fill and granular subgrades can be tested for shear strength using drained or undrained triaxial procedures. (These tests are discussed in greater detail in Sections 2.3 and 2.6). The tests listed under Item 7 of Table 2-15 are recommended here, because the embankment failure has already identified this area of track as one of special importance. For preliminary stability analyses, correlations between soil index parameters and shear strength, as contained in Appendix A, may be helpful, but final analyses should be based on specific engineering property tests. If the magnitude of the problem permits, construction of test sections should be considered to verify and refine designs.

Cost - The total cost of constructing a wider embankment is determined by these five elements: conducting engineering studies, purchasing borrow material, transporting borrow material, placing and compacting the fill, and reconstructing drainage facilities.

The cost of each of these elements is site-specific; however, there are general guidelines. Based on data provided in Building Construction Cost Data-1980 (by Robert S. Means Co.) and on local experience, the cost to buy, load, haul two miles, spread, and compact common borrow fill is about \$4.50 per cubic yard. However, based on availability and ownership of fill, this cost can range from \$3.50 to more than \$10.00. If the borrow haul distance is five miles, the additional cost to haul it the extra three miles is approximately \$1.25 per cubic yard. In addition, the engineering study should be performed by an experienced geotechnical engineer. Even for the most limited embankment failures, the cost for exploration, testing, and engineering will probably exceed \$5,000, and higher costs could be incurred for embankments that are very high or very long.

Discussion - Embankment widening has the following advantages for correcting instability:

1. Frequently, the construction of berms or slopes at the sides of embankments may be accomplished without interrupting train operations or removing the track structure. However, if corrective action is required because of a major failure and loss of the track, the embankment will have to be stabilized--at least temporarily--before continuing train operations. If the slide occurs during construction, the berms may be placed in the same manner as the embankment fill, except material quality and compaction are less critical.

2. Conventional construction equipment and methods are used to construct fills.

3. Based on a reasonably complete engineering study, widening the embankment should succeed in halting or preventing slope failure.

4. The cost of widening the embankment, compared with other methods of stabilization, will be low to moderate.

Embankment widening, however, has the following disadvantages:

1. Embankment widening requires sufficient right-of-way or space alongside the track on which to place the fill. This is frequently the major restriction on implementing the procedure.

2. Placing additional fill adjacent to an existing embankment will increase the stresses beneath the embankment, which will lead to settlement of the embankment due to consolidation of the foundation soils.

3. In designing the berm, the stability of the toe of the berm must first be determined to guard against failure of even the lowered berm-embankment height.

4. Placement of the berm will frequently cover pre-existing drainage facilities, which must then be reconstructed following embankment changes.

Embankment or Slope Lowering

Application - As previously described under Embankment Widening, this type of embankment geometry redesign and reconstruction is applicable to all soil types, with cohesive soils experiencing the most frequent problems.

Description - Embankment lowering involves removing the top portion of an embankment to reduce the weight of the material above the subgrade and foundation, as shown in Figure 5-24. This method is more applicable for cut slopes, where it may take the form of benching (as shown in Figure 5-25) or merely trimming the slope to some uniform, flatter grade. Benching offers an added advantage of stopping material and water from rolling or flowing down the entire length of the slope.

Embankment lowering principally improves stability by reducing the weight of material and driving moment which affects the movement of the slope. Embankment lowering requires excavating the material from the existing embankment or from an adjacent cut slope using conventional earthwork equipment and methods. Following the excavation, the newly exposed track subgrade should be carefully graded to drain and should be compacted from the surface with heavy rollers prior to replacing the subballast and ballast. Newly exposed surfaces can be protected from erosion using loam and seed or by covering them with a layer of gravel or crushed stone that will not wash off. If the top of an embankment that supports track has been lowered, the track structure will have to be rebuilt.

Engineering - The engineering study to evaluate this type of stabilization is generally the same as that described for embankment widening. For cut slopes, the critical case normally is the long-term stability previously described in Section 2.7. When evaluating cut slopes, it is necessary to determine the pore pressure distribution behind the slope; piezometers should be used to measure the pore pressure. Evaluation of drained shear strength parameters of cohesive soils is necessary to forecast the long-term stability condition. Consolidated-undrained triaxial shear tests with pore pressure measurements are often the most effective way to determine both undrained and drained shear strength parameters of cohesive soils. Both drained and undrained stability conditions should be analyzed. In stiff, overconsolidated clays, long-term stability may be controlled by residual strength of the soil.

Cost - The total cost of embankment lowering is comprised of the following elements: conducting engineering studies, excavating and loading the material, hauling and disposing of the material, constructing erosion protection (if required), and reconstructing track (if required). Costs of these factors are variable, depending on details of the site and soil conditions.

For erosion protection, mesh (such as jute or plastic) or seeding, or a combination of both, may be used. The cost for this treatment ranges from about $0.40 to $0.60 per square yard. If track reconstruction is required, the cost of this item may range from $40.00 to more than $100.00 per track foot. The cost can be reduced, however, by re-using rails, ties, etc. Because embankment lowering requires complete rebuilding of the track, it will probably be a practical choice only if rebuilding the track is scheduled for reasons other than embankment stabilization.

The engineering program cost is similar to that described for embankment widening: a budget of at least $5,000.00 should be allocated.

Discussion - The advantages of embankment lowering are as follows:

1. Conventional construction equipment and methods can be used.

2. Cutting back or benching cut slopes can sometimes be accomplished without interrupting traffic.

3. If the slope or embankment redesign is based on a careful geotechnical engineering study, the chance for successful correction of the problem is high.

4. For cut slopes, the cost of lowering the slope or embankment is low to moderate.

Embankment lowering has these disadvantages:

1. Changing the embankment height may require complete realignment of the track grading in a region.

2. Lowering an embankment requires complete interruption of service and reconstruction of the track.

3. Cutting back or benching a slope requires increased rights-of-way or approval from adjacent property owners.

4. If the material removed is contaminated, disposal may pose a significant problem.

5. If a cut slope is modified, stability will generally lessen over time; the new geometry should be monitored in the field after stabilization to see if slope movement has ceased.

Lightweight Fill

Application - If the shear strength of the subgrade and foundation is not able to support the weight of an embankment of a desired height and composed of conventional earth materials, the embankment can be constructed using special lightweight materials to reduce its weight. This method of improving embankment stability, as with the previously discussed methods, can be used for all soil types; however, it will receive most application for soft cohesive soil foundations. Low-density fill also can be used to alleviate settlements of embankments and track caused by consolidation of the subgrade.

Description - Common earth materials used for embankment fill have in-place total unit weights of 110 pcf to 150 pcf. The common range of dry unit weight is 100 pcf to 135 pcf. There are lightweight aggregates available with high porosity or void space that have dry unit weights in the range of 55 pcf to 70 pcf. Lightweight aggregates are materials such as expanded shale, porous slag, compacted fly ash, compacted cinders, and crushed shells. Extraordinarily low unit weight, lightweight concrete can be produced with special foam admixtures to produce a fill with a total unit weight as low as 25 pcf to 50 pcf. Lower unit weights are associated with lower compressive or shear strength materials. For all lightweight materials, shear strength and resistance to degradation are lower than for normal granular fill. Generally, it is good practice to place a blanket of conventional, compacted granular fill between the top of the lightweight fill and the bottom of the track structure to limit the shear stresses and frost penetration in the lower-strength, lightweight material.

Use of lightweight fill increases stability by reducing embankment weight and the driving moment that induces failure. With respect to consolidation, use of lightweight fill decreases the additional stresses applied to the subgrade, thereby lessening the settlement.

Engineering - Lightweight fill is probably practicable for new construction only or for rehabilitation where the fill might be used to raise the track level while limiting embankment weight. As such, the engineering program should be incorporated into the general engineering of a track section. The critical engineering decisions here deal with specifying the thickness of blanket fill between the track structure and the top of the lightweight fill, and designing measures to limit penetration of surface water into the lightweight, porous fill which would increase the unit weight and might lead to degradation by weathering, slaking, or frost action. Design of track structure to limit subgrade stresses is the subject of a companion report of this study. The potential for weathering, slaking, and frost degradation of aggregates is discussed in Sections 3 and 4 of this report.

Cost - The principal costs associated with using lightweight fill are the purchase and hauling costs of the fill material. If the track is located near a metal mill where there is excess porous slag or other suitable material, the cost of lightweight fill may be comparable to conventional, granular fill. Alternatively, there may be costs of $5 to $25 per cubic yard to purchase lightweight aggregate. Foam concrete fill may cost $45 to $60 per cubic yard in place. These costs are site- and project-specific; however, as an alternative to employing costlier structural measures to alleviate problems--e.g., using pile-supported trestles--lightweight fill generally is competitively priced.

Discussion - There are two advantages to using lightweight fill:

1. The standard embankment sections may be maintained.

2. Conventional earthwork equipment may be used with lightweight aggregate fills, although the compactive effort must be limited to prevent the pores from being crushed.

The disadvantages of using the fill are:

1. Complete disruption of the track is required to replace conventional fill with lightweight fill as a rehabilitation practice; as such, lightweight fill may be practical only for new construction or for raising the surface of existing track where disruption of the track is predetermined.

2. The cost of lightweight fill can be negligible to high, depending on local availability and on the desired fill density (i.e., lower densities are associated with higher costs).

3. Special drainage measurements may be required to prevent water infiltration into the lightweight fill and to limit frost penetration.

Retaining Structures

Application - This stabilization measure includes constructing such structures as crib walls or other retaining walls along an embankment or cut slope, tie and pole driving along embankments, and using newer concepts such as Reinforced Earth walls.

Retaining structures have two applications for filled embankments. If the foundation is stable but the embankment is straining internally, the retaining structure can limit lateral spreading of the fill. Spreading is likely to develop if the fill is composed of soft to medium clay, silty or clayey sands and gravel, or a soil of low strength and modulus that probably was placed without compaction in the past. As water invades the loose fill and voids in the fill, there is progressive softening and reduced strength of the embankment. The second application is to retain the fill where space limitations restrict embankment width even though the foundation and fill strength are adequate.

Description - Three types of retaining structures can be used to stabilize railroad embankments and cut slopes--cantilever and tied retaining structures, crib walls, and conventional cast-in-place walls.

Cantilever and tied retaining structures have long been used to stabilize railroad embankments by tie and pole driving. Frequently, two rows of piles are driven into opposite sides of the embankment, and, often, the two rows are tied together with steel rods or other tension members, as shown in Figure 5-26. The piles should be driven to a depth equal to twice the depth of the failure surface, although 1.5 times the depth to the failure surface may be considered the minimum to develop a moment reaction at the bottom of the pile. Timber piles are used most often in lengths of 8 feet to 20 feet. Used, serviceable ties may also be driven to stabilize shallow-seated failures. If a very deep-seated slide is involved and large bending capacity is required, drilled-in steel beam sections encased in concrete, and sometimes socketed into rock, may be used. The piles are driven or cut off about 6 inches to 12 inches below the bottom of the ties to limit interference with maintenance equipment.

The spacing of the ties or piles along the track is about five pile diameters, although spacing ranges from about two to eight diameters, and sometimes a pile is driven at the end of every tie. If close longitudinal spacing is required, the piles are sometimes driven in two rows in a zigzag pattern on each side. Laterally, the piles are driven close to the track, usually 4 inches to 12 inches from the ends of the ties.

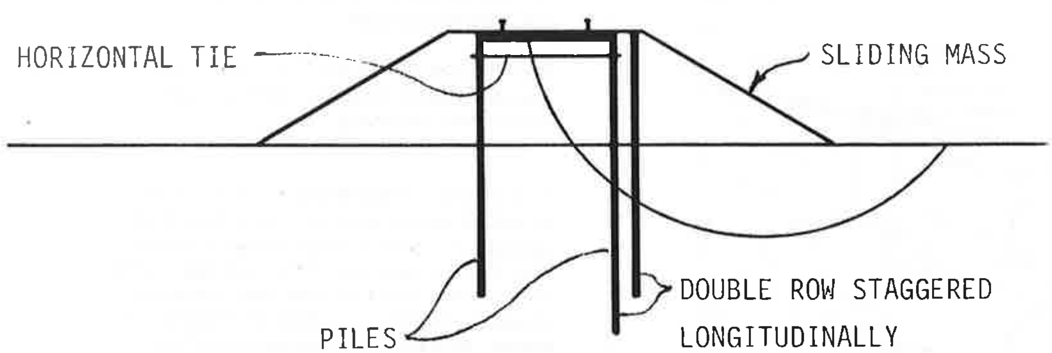
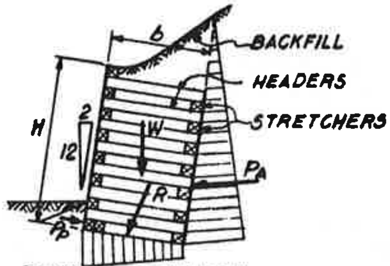
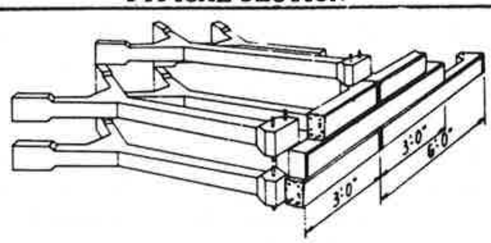


FIGURE 5-26. STABILIZATION BY PILE DRIVING

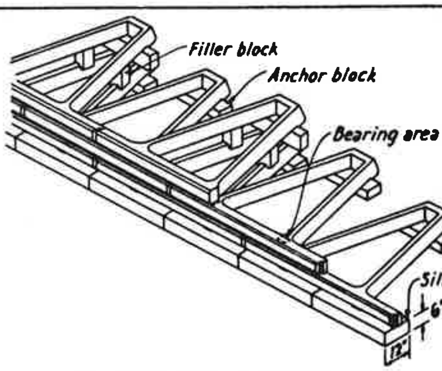
Crib walls are gravity-retaining structures that work by having the weight of the structure supplied primarily by earth fill within cribbing. Cribbing is a box structure of timber, steel, or concrete beams that lock together to form a flexible, porous series of box cells, as shown in Figure 5-27. Precast concrete boxes can also be stacked and filled with earth to form a crib wall. The cells are filled with free-draining, granular material, and the weight of the filled crib is sufficient to retain soil backfill behind one side.



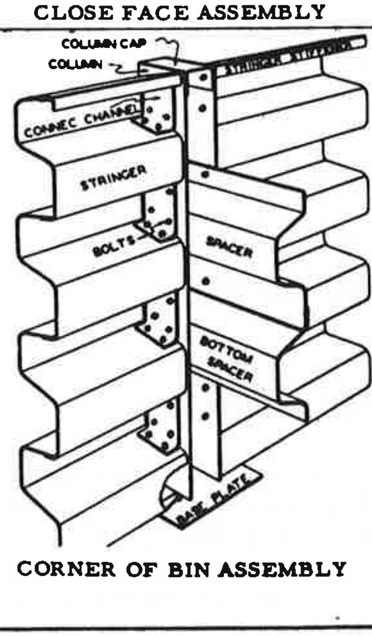
TYPICAL SECTION



FISH TAIL TYPE ASSEMBLY



CLOSE FACE ASSEMBLY



CORNER OF BIN ASSEMBLY

CRIB RETAINING WALLS

TYPES - Common types of cribs shown on accompanying diagrams.

CRIBBING MATERIALS - Timber, concrete, and metal.

FILL - Crushed stone, other coarse granular material, including rock less than 12-in. size.

DESIGN - Design criteria for gravity walls apply. Wall section resisting overturning is taken as a rectangle of dimension (H x b).

Weight of crib is equal to that of material within (H x b), including weight of crib members.
Low walls (4 ft high and under) may be made with a plumb face. Higher walls are battered on the face at least 2 in. per foot. For high walls (12 ft high and over) the batter is increased or supplemental cribs added at the back.
In open face cribs, the space between stretchers should not exceed 8 in. so as to properly retain the fill.
Expansion joints for concrete and metal cribbing are spaced no more than 90 ft.

FILLING - The wall should not be laid up higher than 3 ft above the level of the fill within the crib.

DRAINAGE & FROST ACTION - No special requirements, wall should be made free draining.

BIN TYPE RETAINING WALL - Composed of metal bins or cells joined to special columnar units at the corners. The design requirements are the same as for crib walls except that suitable drainage behind the wall is needed. Internal stresses are investigated in accordance with criteria for cellular walls.

Reproduced from Soil Mechanics, Foundations, and Earth Structures, p. 7-10-16, U.S. NAVFAC Design Manual DM-7. Year of first publication: 1961.

FIGURE 5-27. CRIB TYPE RETAINING STRUCTURES

Crib walls may be used to help support embankments or cut slopes. As shown in Figure 5-28, the crib wall provides lateral confinement for a spreading fill. For the slide of the cut slope, the crib wall permits construction of a stabilizing berm within a limited area, retaining the slope and preserving the existing drainage channel.

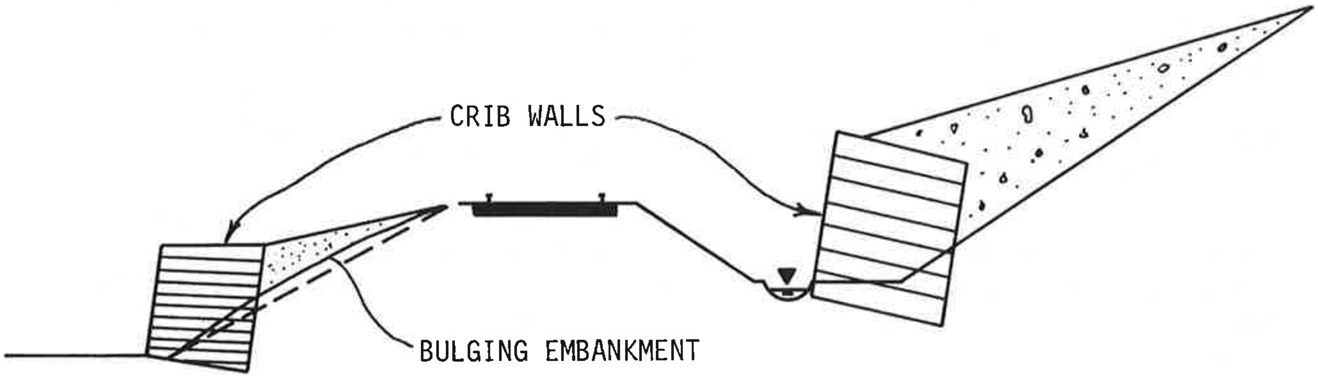


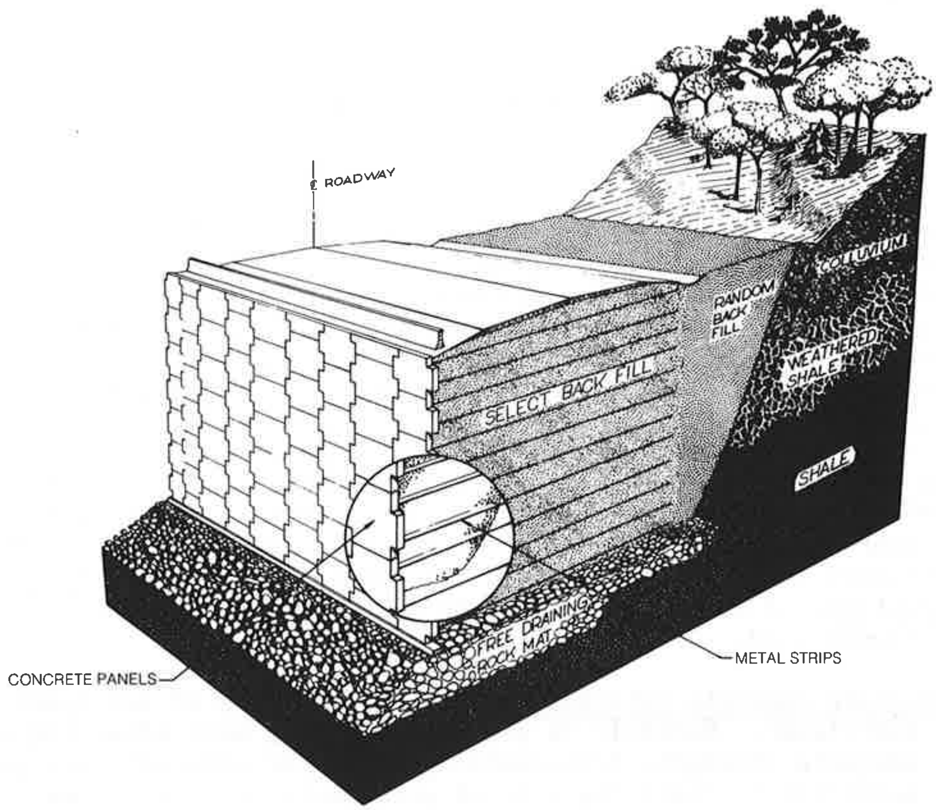
FIGURE 5-28. SUPPORT BY CRIB WALLS

A relatively recent development (1964) is a retaining structure system called Reinforced Earth.® In this system, strips of sheet metal, galvanized steel, or aluminum are attached to prefabricated wall facing. The strips are not attached to any member buried in the backfill but are held by friction with the compacted, granular, backfill soil. The length of strips is about equal to the maximum height of the wall, and they are embedded in the fill every 0.3m (1 foot) vertically with horizontal spacing of about 0.8m (2.5 feet). The interaction between the metal strips and the earth backfill forms a reinforced earth mass to support a slope or embankment. The wall-facing elements, which can be either steel or precast concrete, prevent sloughing of the soil near the edge of the wall. A complete description of Reinforced Earth is contained in a 1975 report by J. L. Walkinshaw entitled Reinforced Earth Construction. A conceptual sketch of a Reinforced Earth structure is shown in Figure 5-29.

Cast-in-place concrete retaining walls generally are of two types: gravity and cantilever. Backfill is placed behind the wall after the concrete has reached adequate strength. Free-draining, granular materials are preferred to cohesive materials to limit the rise of groundwater behind the wall. Drainage pipes, which are frequently installed through the walls to limit the rise of water behind them, are essential for successful performance. Sketches of conventional retaining walls are shown in Figure 5-30.

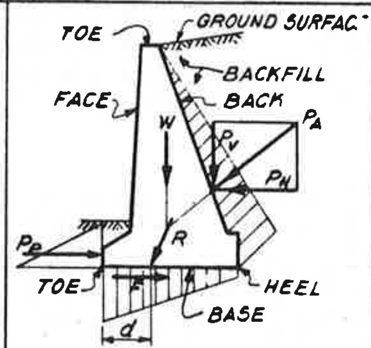
Engineering - The scope of engineering study recommended to design a retaining structure may be quite varied, depending on the structure. Cantilever and tied retaining structures constructed by pole driving have frequently been used on railroads for emergency stabilization with limited subsurface exploration. The principal design parameter is the depth of the sliding surface. In a 1974 paper entitled, "Roadbed Stabilization - Various Methods," J. B. Farris suggests, "A rule of thumb in considering treatment for subgrade failures is that the distance of the failure plane beneath the top of the rail is approximately equal to the distance between the near rail and the top of the heaving ground." When it is paramount to correct unstable track quickly, this rule of thumb may be satisfactory to determine pile depth. Horizontal spacing is based either on previous experience in an area or on trial and error. Additional piles should be installed if movements are not arrested.

For more elaborate and costly types of structures, such as crib and cast-in-place retaining walls, a complete geotechnical engineering study should be carried out and should include exploration, testing, and analysis. Analysis of crib and retaining wall structures are described briefly in Figures 5-27 and 5-30.



Reproduced from "Construction of a Reinforced Earth Fill Along Interstate 40 in Tennessee," by D. L. Royster. Year of First publication: 1974.

FIGURE 5-29. CONCEPTUAL SKETCH OF REINFORCED EARTH STRUCTURE

<table>
<tr><th>Type of wall</th><th>Load diagram</th><th>Design factors</th></tr>
<tr><td>GRAVITY</td><td>
</td>
<td rowspan="4">
LOCATION OF RESULTANT

Moments about toe:
$$d = \frac{Wa + P_V e - P_H b}{W + P_V}$$
Assuming $P_P = 0$

OVERTURNING

Moments about toe:
$$F_S = \frac{Wa}{P_H b - P_V e} \geq 1.5$$
Ignore overturning if R is within middle third (soil), middle half (rock). Check R at different horizontal planes for gravity walls.

RESISTANCE AGAINST SLIDING
$$F_S = \frac{(W + P_V)\tan\delta + C_a B}{P_H} \geq 1.5$$
$$F_S = \frac{(W + P_V)\tan\delta + C_a B + P_P}{P_H} \geq 2.0$$
$$F = (W + P_V)\tan\delta + C_a B$$
For coefficients of friction between base and soil see Table 10-1.

C_a = adhesion between soil and base

$\tan\delta$ = friction factor between soil and base

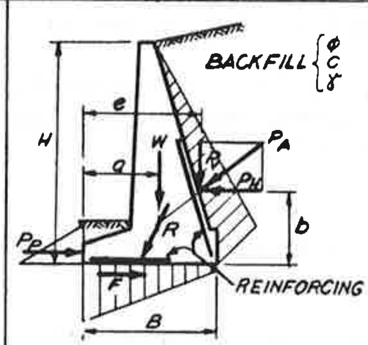
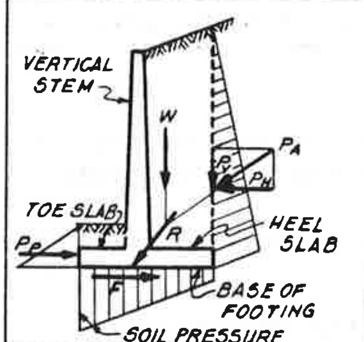
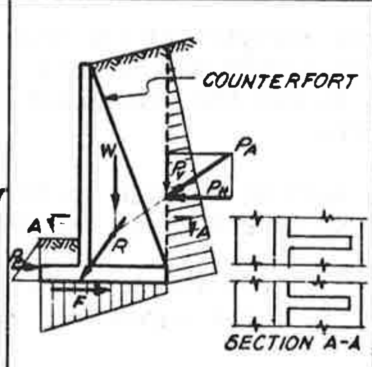
W = Includes weight of wall and soil in front for gravity and semi-gravity walls. Includes weight of wall and soil above footing, for cantilever and counterfort walls.

CONTACT PRESSURE ON FOUNDATION

For allowable bearing pressure for inclined load on strip foundation, see Ch. 11.

For analysis of pile loads beneath strip foundation, see Ch. 13.

OVERALL STABILITY

For analysis of overall stability, see Ch.7.
</td></tr>
<tr><td>SEMI-GRAVITY</td><td>
</td></tr>
<tr><td>CANTILEVER</td><td>
</td></tr>
<tr><td>COUNTERFORT</td><td>
</td></tr>
</table>

Reproduced from Soil Mechanics, Foundation, and Earth Structures, p. 7-10-13, U.S. NAVFAC Design Manual DM-7. Year of first publication: 1961.

FIGURE 5-30. EXAMPLES AND DESIGN CRITERIA FOR CAST-IN-PLACE RETAINING WALLS

Cost - The cost of using retaining structures is comprised of costs associated with purchasing retaining structure materials and installation; backfilling the structures; correcting the grading of adjacent slope(s) or embankment(s); and completing engineering studies. For pole driving, the cost of a 20-foot, creosote-treated timber pile is approximately $175 to $200 each in place. Driving ties to a depth of 8 feet costs about $30 to $50 each, depending on the value of the ties used.

For a wall 10 feet high, a steel-bin crib wall costs approximately $20 per square foot of wall, plus the cost of backfill. A Reinforced Earth® wall costs about $13 to $18 per square foot plus the cost of backfill. A 10-foot-high gravity or cantilever concrete retaining wall costs about $18 to $25 per square foot of exposed wall. Backfill with select, free-draining, granular fill in lifts costs approximately $8 to $10 per cubic yard, but may vary considerably, depending on the supply of acceptable fill material.

All of these costs are affected by shipping costs of materials and accessibility to the site. If working space is severely restricted, the requirement for hand labor will inflate the cost significantly.

Discussion - For pole and tie driving, the principal advantages are as follows:

1. The structure can be installed with minimal planning. Materials are often on-hand in maintenance yards, making the method well suited for emergency repairs. Finally, pile driving equipment required is generally available.

2. The cost of this measure is moderate. In tie driving, use is made of materials that may be unsuitable for other track applications.

3. Piles may be installed by on-track or off-track equipment with minimal disruption of operations.

The principal disadvantages of pole and tie driving are:

1. Unless a careful engineering study and analysis of the stabilizing support provided by the piles is conducted, the method may fail to arrest movements. However, additional piles or ties across the embankment can be installed later to increase stabilizing support.

2. Installing piles in a soft, cohesive soil will cause an increase in pore pressure that will temporarily lower the stability of the embankment.

3. If pole driving fails to stabilize the slide, the installed poles will interfere with any subsequent attempt to treat the embankment failure.

The main advantages of crib walls are:

1. Wall elements are standard pieces that fit together into many configurations. Thus, a supply of crib elements may be stocked and adapted to a problem site when required.

2. The crib can accept considerable distortion without failure, so that foundation support need not be rigid.

3. The bulk of the material in the structure is free-draining backfill. This is an advantage if readily available locally, or a disadvantage if such material is scarce.

4. If the design is based on a comprehensive engineering study, the probability of successful installation is high.

5. Assembly of the structure requires little specialized equipment or training.

The disadvantages of crib walls are:

1. Space is required for construction, which may disrupt train operations or even the track structure.

2. The cost is moderate to high.

3. Adequate drainage of the backfill is paramount for successful installation and continued stability of the wall.

Conventional cast-in-place retaining walls have similar advantages and disadvantages to crib walls, with the following differences:

1. A relatively unyielding foundation is required, sometimes necessitating the use of pile support on soft foundations.

2. The cost is relatively high.

3. Construction is time-consuming.

4. Somewhat less construction working space is required than for crib walls.

Finally, an unstable embankment or cut slope must be evaluated from an overall viewpoint. If a retaining structure is installed to halt one mode of failure, a deeper-seated mode may become critical, with the entire retaining structure moving as one rigid body.

Vertical Reinforcement

Application - If a railroad embankment suffers from stability failure or progressive settlement over time, due to yielding or consolidation of the subgrade (leading to degradation of track surface), the condition can be improved by installing vertical compression reinforcement through the embankment. The vertical reinforcement accepts the load of the embankment and any superstructure loads and transfers them through the yielding subgrade to a suitable bearing stratum. The reinforcement may also increase the capacity of the in-situ soil, so that it can accept loads with smaller displacement. Vertical reinforcement is most likely to be used with subgrades comprised of soft clay and plastic silts (USC codes CL, CH, ML, MH) and loose, silty or clayey sands (SM, SC).

To be cost-effective, vertical reinforcement should be used where the soil profile contains a shallow layer of yielding or compressible material underlain at moderate depth (10 feet to 40 feet) by a suitable, bearing stratum. A typical soil profile that is suitable for stabilization by vertical reinforcement might be 10 feet of soft clay and organic silt underlain by compact sand.

Description - Three types of vertical reinforcement have potential application for stabilization of railroad track: piles, lime columns, and stone or sand columns. Each is described below.

Piles have been used in Sweden to support highway embankments over very soft subgrades such as peat. The piles are driven through the compressible stratum to a suitable bearing stratum. Pile caps are then built over the piles, and the embankment is constructed over the pile caps. The caps are not continuous, but, rather, arching in the embankment soil spans the gaps between the pile caps.

Augered concrete piles called post-hole piles have been used to stabilize airfield pavements in the United States. A solid auger that is 150m to 300mm (6 inches to 12 inches) in diameter is used to penetrate the pavement and the subgrade. Piles are formed to depths of 2m to 3m (6 feet to 10 feet); the maximum depth that can be reached depends on the ability of the soil to maintain the open, augered hole without support. After the hole is free of water and loose soil, conventional concrete is poured into it. Hole spacings of 3 feet and 5 feet were evaluated at the Altus AFB Soil Layer Stabilization Test Sections, as reported by Thompson and Robnett in 1975.

Root Piles® are a system of rotary or compression drilled, cast-in-place, reinforced concrete piles that are installed to stabilize moving slopes or to support foundations. The system was developed in the 1960's in Italy by Fondedile, S.p.A. Usually small in diameter (i.e., 3.5 inches to 12 inches), the piles are drilled in an interwoven network pattern to form a unified mass of piles and soil or "reinforced soil." After installation of bar or cage reinforcement, concrete is placed in the pile under pressure of up to

2 MPa (300 psi), causing the bond between the concrete and soil to be high and the soil surrounding the concrete to densify. Fracturing of the soil may occur, resulting in an irregular pile cross section that increases pile side friction.

Root Piles have been used most frequently to vertically reinforce foundations of existing structures. The piles are drilled through existing footings and are connected to the footings by the high-pressure placement of the concrete. Although not mentioned in the literature researched for this report, Root Piles might be used to stabilize embankment settlements in a similar manner. The small diameter piles could be installed without removing the track structure. The pressure placement of the concrete would engage the embankment fill with the piles so that no pile cap would be required and surrounding soil would be densified.

Root Piles may also be used to stabilize an embankment or slope failure by one of several mechanisms described by F. Lizzi in his 1978 paper, "'Reticulated Root Piles' - To Correct Landslides." The Root Piles unify the deforming mass of soil, restricting displacements within the mass. Over stiff soils, the piles can provide shear reinforcement to restrict displacements on a mobilized sliding surface. In loose soil, the reticulated Root Pile-soil mass is designed to form a gravity-retaining structure, of soil and piles, supporting soil above the structure. This is similar in concept to a Reinforced Earth®, gravity-retaining structure, except that the Root Piles can be installed without excavation and backfill which is required for the Reinforced Earth.

Lime columns reinforce the subgrade by producing vertical cylinders of lime-stabilized soil, as reported in 1979 by Broms and Boman in the paper "Lime Columns -- A New Foundation Method." Here, an auger is used to mix unslaked lime and soil in-situ to form vertical columns of stabilized soil that stiffen the entire soil mass. The auger is more like an egg beater than a screw because no soil is withdrawn. The auger is 0.5m to 1.5m (20 inches to 60 inches) in diameter and has been used for an average treatment depth of 10m (33 feet), although treatment to 30m is possible. The auger is rotated and inserted to the required depth. Then, during withdrawal, unslaked lime, usually 5 percent to 8 percent of the dry weight of soil, is injected from the bottom of the hollow-stem auger by air pressure and is mixed with the soil by the auger.

The lime columns stabilize the subgrade three ways: (1) the soil-lime columns themselves have bearing capacity greater than the surrounding soil; (2) the unslaked lime draws water from the soil surrounding the column and generates heat while slaking; and (3) the lime columns have higher permeability than the surrounding soil and serve as vertical drains that accelerate consolidation. These effects increase the strength and stiffness of the soil surrounding the columns.

Stone columns or Vibroreplacement are a variation of the Vibroflotation method discussed in Section 5.2. In addition to soft clays and silts (USC codes CL, CH, ML, MH), stone columns are applicable to granular soils containing more than 20 percent fines (SM, SC, GM, GC). (Vibroflotation can only be used to densify clean sands.) A Vibroflot® probe is used to penetrate the loose soil by jetting it with water. After reaching the required depth, the probe is withdrawn, with the hole remaining open due to the pressure of the standing water. The hole is then backfilled in layers about 1m thick with coarse, granular material, which frequently is crushed stone, thus the name "stone columns." By reinserting the probe and vibrating it, the backfill is compacted, and some of the soil surrounding the column is densified by displacing the stone backfill laterally. The resulting stiff columns of stone provide vertical reinforcement of the loose soil. In addition, the columns provide vertical drains that accelerate consolidation of the soil(1).

The Vibroflot is 0.3m to 0.4m in diameter and 4.6m long. Due to the jetting and displacement of the backfill, the resulting column is 0.6m to 1.0m in diameter. The Vibroflot rig normally operates to a depth of 14m, but may be cable hung from a crane to operate at greater depths. Column spacing usually is 1.2m to 2.7m.

Stone columns have been used to form the foundation of buildings as well as highway embankments. In one case, a Reinforced Earth wall was supported on a stone column-stabilized foundation to limit the width of the highway fill over the stone columns(2).

Sand columns have been used in the past to stabilize clay subgrades beneath railroad track. A 1948 report by AREA Committee 1 on "Roadbed Stabilization" reported that the Southern Railway used 12-inch by 12-inch timbers to form holes in the subgrade. In this case, 12-inch-square spuds were driven to a depth of 6 feet to 8 feet directly between in-place ties. Once the spud was withdrawn, the open hole was filled with sand and tamped. Three or four rows of holes were driven into each crib, and a row of holes was driven at the end of each tie. This spacing provided for replacing about 20 percent of the subgrade area with sand.

Sand compaction piles may be installed with a casing for use in cohesionless soils that will not stand open unsupported. Sand is injected from the bottom of the casing by air pressure. Compaction can be by vibratory hammer or by a drop pile hammer. The casing is about 0.4m (16 inches) in diameter, and the columns generally are spaced 1.5m to 2.2m (5 feet to 7 feet) center-to-center. The maximum treatment depth is about 13m (43 feet).

---

(1)K. Engelhardt et al., "Vibroreplacement - A Method to Strengthen Cohesive Soils In-Situ," Reprint Paper 2281, ASCE National Structural Engineering Meeting, Cincinnati, Ohio, April 1974, pp. 3-5.
(2)A. Munoz and R.M. Mattox, "Vibroreplacement and Reinforced Earth Unite to Strengthen a Weak Foundation," Civil Engineering, May 1977, pp. 59-60.

Engineering - The exploration and testing procedures set forth in Section 2.1 are generally satisfactory for designing vertical reinforcement stabilization. It will be necessary to take explorations to extra depth, since the support of the track is being transferred to a deeper level. The properties of the embankment are important, because they affect the interaction of the reinforcing elements with the embankment soil.

For the lime column method, the lime reactivity of the soil should be determined to decide on the quantity of lime to be added, as discussed in Section 5.4. Shear strength tests of lime-stabilized samples will aid in evaluating the strength of the lime columns.

Many of the design/analysis procedures for these methods are greatly influenced by prior experience. In 1976, Goldberg et al., in Volume III of the report, Lateral Support Systems and Underpinning, set forth load test results on Root Piles®. The 1978 ASCE report, Soil Improvement, provides some additional data on design. The contractors who specialize in these methods have developed their own proprietary designs.

For preliminary analysis, capacity of piles can be estimated by conventional means described in foundation engineering textbooks. Transfer of load from embankment fill to pile caps can be estimated by the empirical rule that arching will transfer the load from the prism of soil above each cap contained within lines inclined 60 degrees from the horizontal. A more rational analysis may also be made using relations for the uplift capacity of buried spread footings. This analysis points out the importance of specifying a high density of backfill in the embankment to achieve a high strength and wider pile cap spacing. However, this and all other analyses should be tempered by experience and may require field tests.

Cost - It is impractical to try to provide specific cost data for these specialized methods. Some of the stabilization techniques have never been used in North America so that pertinent cost data do not exist. All of these methods are considered relatively costly. Even for rough estimates, it is necessary to contact specialty contractors with experience in performing the special techniques.

Discussion - Because the methods of vertical reinforcement of track subgrade are all relatively expensive, they are practicable only for correction and prevention of major foundation problems. However, compared with some other methods, such as excavation/replacement or trestle support, these methods may be preferred.

Design and track performance predictions made after stabilization by these methods can be based only on experience, which is limited. Vertical compressibility of the soil treated by several of these methods decreases by a factor of 1.5 to 5, as reported in the 1978 ASCE report, Soil Improvement. The complex nature of the interaction among the reinforcing elements, the in-place soil, and the supported structure makes definitive predictions of displacements impossible.

Pile reinforcement for embankment support may be used over any type of subgrade soil. Sand and stone columns are more effective in silty and clayey sands, but may also be applied in purely cohesive soils. Lime columns are only effective in treating soft, clayey soils; in particular, those with an in-situ water content greater than the liquid limit.

Root Piles®, post hole piles, and sand columns produced by spud-driving can all be installed without removing the track. It might be possible to develop a lime column rig to operate between the ties, but the diameter of existing equipment is too large. The remaining methods would all require removal of the track superstructure. To use pile support of an embankment, it is necessary to construct or reconstruct the embankment after installation of the piles and pile caps.

Excavation/Replacement

Application - If an unsuitable deposit of soft soil underlies a proposed track route, the subgrade and foundation can be stabilized by removing the unsuitable material and replacing it with suitable fill. For this method to be practical, the following conditions are required:

a. The unsuitable soil must extend only to a limited depth, preferably above the groundwater table.

b. A place must be available to dispose of the soil.

c. Suitable replacement borrow must be economically available.

The types of subgrade soils for which this method generally has been applied are soils that cannot be stabilized economically in-situ, including organic soils, such as peat and organic silt (USC codes Pt, OL, OH), and very soft clays and silts (CL, CH, MH). Because this method requires removal of the natural subgrade, it is practical only for new construction or, more rarely, for reconstruction.

Description - Unsuitable soils can be removed by machine excavation or by displacement of the soil with embankment fill.

Machine excavation is most frequently carried out by backhoes, draglines, or bucket loaders. The generally soft subgrade conditions make operation of self-propelled scrapers impractical. If the subgrade beneath the soil that is being removed also has low stability, only a backhoe or dragline should be used. This will prevent equipment from operating on and disturbing the sensitive, final, excavated subgrade.

Soft soil deposits requiring excavation and replacement usually exist at or below the water table. Therefore, excavation will require dewatering. Often, pumping from sumps within the excavation is used; dewatering with wells or wellpoints is uncommon except in confined areas. It is usually unnecessary to achieve a completely dry bottom--merely one dry enough to confirm that all unsuitable soil has been removed. In some cases, excavation is carried out when the soil is under water. However, unless there is a way to determine satisfactory removal of all unsuitables, the potential still exists for some material to remain that may lead to future settlements or slides.

Backfill of the excavation is best accomplished when the excavation is dry. Fill should be compacted in lifts to a minimum dry density of 95 percent of the maximum Standard Proctor dry density (ASTM D698). Suitable provisions for placement of fill are contained in Section 1.3.5 of the AREA Manual.

Where fill is to be placed below the natural groundwater table level, it may not be practical to adequately dewater the excavation to permit dry placement and compaction of fill. In this instance, fill may be placed in standing water provided that a well-graded, clean sand and gravel (less than 8 percent finer than the No. 200 sieve) or crushed, washed stone material is used. Fill may be placed under water in lifts up to 1m to 2.5m (4 feet to 8 feet), followed by rolling with a heavy, vibratory, steel drum roller with 15 to 20 passes for each lift. Thicker lifts may be used if followed by deep densification, as described in Section 5.2.

Displacement of unsuitable soil can be accomplished if the soil is very soft. The embankment fill is advanced across the area of unsuitable soil, and the fill is constructed to a sufficient height so that the advancing slope of the fill induces a shear failure in the foundation. The embankment fill then settles and the subgrade soils are displaced, enabling the fill materials to rest directly on suitable foundation soils. This method does not provide the assurance that all unsuitable soils are removed; therefore, the performance level of the embankment may not be as high as for the machine excavation procedure. However, displacement removes the requirements for transporting the excavated soil and for dewatering the excavation. It is generally impossible to compact the embankment during placement, although densification after displacement may be carried out.

A variation of the above method is to use explosives to supplement the displacement action of the fill. The explosives may help remove a greater fraction of the unsuitable soil and may permit use of a lower initial embankment height. In any case, the dumped embankment should at least be compacted near the surface and topped by two or more feet of conventionally placed, compacted granular fill.

Engineering - The program of exploration should be sufficient to define the vertical and lateral limits of unsuitable soil. An accurate definition of the groundwater level should be made with observation wells to help evaluate dewatering requirements. If the displacement excavation method is considered, the shear strength of the soft soil should be determined either by laboratory or in-situ tests. These data are used in stability analyses to determine the embankment height required to achieve displacement. Since the success of this method cannot be observed from the surface, test borings through the embankment fill should be carried out to evaluate the effectiveness of the displacement method to remove all unsuitable soil. Further discussion is provided by Terzaghi and Peck(1).

Cost - The costs for excavation/replacement methods can be estimated, based on conventional construction experience, using data such as those provided by the Robert S. Means Co. Excavation by machine costs \$1 to \$2 per cubic yard and will depend on the size of the machine and access to the work. Hauling costs for disposal range from \$1 to \$2 per cubic yard, depending on distance, and the cost of dewatering depends on the groundwater level and permeability of the soil. For a modest-sized excavation, the cost of pumping from sumps is \$100 to \$500 per day. Dewatering costs are also affected by the care used in protecting the final subgrade from disturbance by water seepage. Dewatering from wells is more expensive.

Backfill and compaction of fill in place will cost \$5 to \$10 per cubic yard or more, depending on availability of suitable material. Placement of fill underwater may result in a decreased cost due to decreased labor, but it requires high-quality fill that may be associated with a premium cost.

In using a displacement method, there is no cost associated with excavation. However, this method requires extra fill to be placed because of losses during the slope failure. If explosives are used, the cost of drilling the holes and placing the charges must be added. This is specialized work for which general costs cannot be provided.

Discussion - Where subgrade soil cannot support track, excavation and replacement is probably the most common method used to stabilize the subgrade for new construction, because it is cost-effective, special equipment or materials is not required, and because it is the most economical for locations where the bottom of unsuitable soil is shallow, usually 2.5m to 4m (8 feet to 12 feet) maximum.

In addition to depth of unsuitable soil, the depth of the groundwater and grain-size characteristics of the subgrade beneath the soil are the principal factors influencing the cost and choice of method.

---

(1)K. Terzaghi and R. Peck, Soil Mechanics in Engineering Practice, 2nd edition, John Wiley & Sons, New York, 1967, pp. 469-471.

Displacement can be used only in open areas where the displaced, soft soil will not interfere with adjacent property. In general, displacement has been used only where the depth of unsuitable soil has made mechanical excavation impractical. Excavation depth becomes more important as the difficulty and cost of dewatering increase. A factor complicating this method is the potential occurrence of a strong layer over the soft foundation. In some cases, a thick root mat will overlie organic deposits. This mat can restrict displacement, requiring extra height of fill to induce failure and displacement.

5.6 DESIGN OF STABILIZATION PROGRAMS

Any design effort involving subgrade stabilization requires the following steps:

a. Exploration and identification of existing conditions.

b. Quantification of performance characteristics and material properties.

c. Identification of subgrade problems and deficiencies.

d. Identification of acceptable stabilization methods; development of preliminary designs.

e. Evaluation of alternatives; selection of optimum stabilization method considering factors such as cost, interruption, reliability, construction time, flexibility to respond to unforeseen conditions, and collateral scheduled or unscheduled maintenance activities.

f. Development of detailed plans and specifications.

g. Observations during construction; design modifications.

h. Post-stabilization performance monitoring.

Because of the great number of factors to be considered in steps "d" and "e" of the design of a stabilization program, it is impossible to provide procedures or guidelines that may be applied generally. However, as a preliminary guide, stabilization methods can be categorized according to the groups of subgrade soils and types of subgrade problems that can be improved or treated. This type of categorization is provided in Tables 5-9 and 5-10. These tables are helpful in carrying out step "d" of the design process. Frequently, the optimum stabilization program will combine several methods, and special problems may be treated best by some special measures not discussed herein.

TABLE 5-9. APPLICATION OF STABILIZATION METHODS TO SUBGRADE PROBLEMS

STABILIZATION METHOD	SUBGRADE PROBLEMS											
	STABILITY	CREEP	CONSOLIDATION	SURFACE SLOUGHS	MUD PUMPING	SQUEEZES	BALLAST POCKETS	FROST ACTION	SWELLING	COLLAPSE	LIQUEFACTION	EROSION
DRAINAGE	X											
Lateral Drains				X	X	X	X	X	X		X	
Interceptor Drains	X			X				X				X
Cross Drains				X	X	X	X	X	X		X	
Horizontal Drains	X						X					X
IN-PLACE MODIFICATION												
Grouting	X	X		X			X			X	X	X
Lime Slurry Injection	X	X		X	X	X	X		X	X		
Deep Densification										X	X	
Preloading	X	X	X							X		
Prewetting									X	X		
Salting								X	X			
Electrochemical	X	X	X		X	X			X			
LAYER INSERTS												
Subballast				X	X	X		X	X			
Filter Fabric					X	X			X			
Impermeable Membrane	X							X	X			X
Insulation								X	X			
Capillary, Clay Interrupt								X				
COMPACTION			X	X		X				X	X	
Cement				X	X	X		X	X	X	X	X
Lime				X	X	X			X	X	X	
Bitumen				X	X	X		X	X		X	X
EMBANKMENT STABILIZATION												
Change Geometry	X	X	X								X	X
Retaining Structures	X	X										X
Vertical Reinforcement	X	X	X								X	
Machine Excavation	X	X	X					X	X	X	X	
Displacement	X	X	X									

TABLE 5-10. APPLICATION OF STABILIZATION METHODS TO TREAT SUBGRADE SOIL TYPES

STABILIZATION METHOD	UNIFIED SOIL CLASSIFICATION GROUP														
	GRAVEL				SAND				LOW PLASTIC			HIGH PLASTIC			
	GW - WELL GRADED	GP - POORLY GRADED	GM - SILTY	GC - CLAYEY	SW - WELL GRADED	SP - POORLY GRADED	SM - SILTY	SC - CLAYEY	CL - CLAY	ML - SILT	OL - ORGANIC	CH - CLAY	MH - SILT	OH - ORGANIC	PT - PEAT
DRAINAGE															
Lateral Drains	X	X	X	X	X	X	X	X	X	X	X	X	X	X	X
Interceptor Drains			X	X	X	X	X	X	X	X	X	X	X	X	X
Cross Drains			X	X			X	X	X	X	X				
Horizontal Drains			X	X	X	X	X	X	X	X	X				
IN-PLACE MODIFICATION															
Grouting		X	X	X		X	X			X					
Lime Slurry Injection				X				X	X	X		X	X		
Deep Densification		X	X	X	X	X	X	X		X					
Preloading									X	X	X	X	X	X	X
Prewetting										X	X	X	X		
Salting			X			X	X		X	X	X		X		
Electrochemical												X	X	X	
LAYER INSERTS															
Subballast		X	X	X	X	X	X	X	X	X	X	X	X	X	X
Filter Fabric			X	X	X	X	X	X	X	X	X	X	X	X	X
Impermeable Membrane			X	X			X	X	X	X	X	X	X	X	
Insulation			X	X		X	X	X	X	X	X				
Capillary, Clay Interrupt			X	X		X	X	X	X	X	X				
COMPACTION	X	X	X	X	X	X	X	X	X	X	X	X	X	X	
Cement		X	X	X		X	X	X	X	X		X	X		
Lime				X				X	X	X		X	X		
Bitumen		X	X			X	X	X	X	X	X	X	X	X	
EMBANKMENT STABILIZATION															
Change Geometry									X	X	X	X	X	X	X
Retaining Structures			X	X			X	X	X	X		X	X		
Vertical Reinforcement									X	X	X	X	X	X	X
Machine Excavation									X	X	X	X	X	X	X
Displacement									X	X	X	X	X	X	X

In some cases, the cause and cure of substructure failure is clear. However, investigation of problem situations should always be performed carefully in order that subtle features are not overlooked. The services of a geotechnical specialist with knowledge of soil behavior, identification of soil failure mechanisms, and familiarity with the substructure and performance of railroad track should be sought in many cases. Subgrade stabilization measures are usually expensive. Any stabilization program should be instituted only after a thorough understanding of the substructure problems and of how a particular stabilization method or methods will cure the problems.

6. CONCLUSIONS

In conventional railroad track--in which the rails are fastened to individual crossties--earth materials are used to construct ballast and subballast layers. Together, the ballast, subballast, and subgrade comprise the track's substructure, with the rails, fasteners, and ties the superstructure. All these elements interact to provide a track with a set of performance characteristics that affect the operation of trains.

To provide suitable support and a guideway for train operations, track geometry should be set following specifications that are appropriate for a desired operating speed. Over time, the track is expected to retain this geometry, although it will be subject to stress due to train loading and the environment.

The engineering of a railroad track's substructure should be aimed toward providing a substructure that readily permits maintenance operations to set the desired initial track geometry, yet limits the track displacements induced by the substructure elements. In this report, the properties and use of earth materials in the structure of conventional railroad track and stabilization of track subgrades have been discussed. The review of pertinent literature written by practicing engineers and researchers has included railroad engineering principles, as well as such fields as highway engineering, foundation engineering, concrete technology, geology, and soil and rock mechanics. This has been combined with personal communications with practicing railroad engineers and general experience with earth materials engineering.

Earth materials have been used in railroad track engineering for more than 150 years. However, there have been few instances where the performance of these materials has been systematically studied in track. Most of the published research on ballast and subballast has appeared in the last decade. But, earth materials are used in all civil engineering structures. To date, there is ample experience documented on the use of earth materials in other fields of civil engineering, and this experience is transferable to track engineering practices. Based on this information, preliminary recommendations on the use of earth materials in track engineering have been developed. However, to realize the ultimate goals of developing a definitive guide for selecting earth materials for track construction and predicting track maintenance requirements, considerable additional research is needed. In particular, systematic evaluations of the performance of earth materials in actual track installations would lead to significant advances in the reliable and economical selection of earth materials for track construction. Ultimately, these studies, coupled with investigations of the laboratory performance of materials, will lead to more accurate methods of predicting service life and required frequency of maintenance in order to keep track geometry and displacements within acceptable bounds.

Currently maintenance, rehabilitation, and upgrading of existing track are far more actively carried out in North America than is construction of new track. For this reason, track subgrade stabilization methods have great practical significance to the industry. Many techniques with potential application to railroad track stabilization have been described. Some of these, including use of filter fabric, drainage improvements, and cement and lime treatment of compacted layers, have been used by railroads and systematically studied so that general design and use criteria are available. Some techniques, including cement pressure grouting of slides and lime slurry pressure injection, have been used by railroads with limited understanding of the stabilization/improvement process. The effectiveness of these applications has been erratic. Finally, there are a great number of available stabilization methods that have been used in applications, such as highways and building structures, but have never been applied to railroad track.

Development of guidelines for subgrade stabilization will require imaginative application of the available stabilization methods and systematic evaluation of the performance of stabilization projects. A prerequisite for developing this type of evaluated experience is to gather geotechnical data on the substructure characteristics at sites of stabilization programs, including subgrade soil classification, groundwater conditions, and other factors discussed in this report. Moreover, measurements of changes in track performance and maintenance requirements will provide the basis of benefit and cost evaluation of stabilization for future consideration.

In summary, there are a great number of available tools and substantial experience that might be adopted for use in railroad engineering practice. Testing of these applications on actual railroads will require additional years of experience. But it is probable that some immediate benefits would be realized from adoption of some of the available technologies described in this report. In addition, outlines of suggested testing and evaluation procedures for work with subgrades, ballast, and subballast are provided. These suggested practices offer the opportunity for substantial future railroad technology developments and benefits. It is hoped that the material presented herein will be considered by the industry for implementation, in-service testing, and development of guidelines for substructure materials engineering.

APPENDIX A

CORRELATIONS BETWEEN INDEX PARAMETERS AND ENGINEERING SOIL PROPERTIES

Section 2.4 of the text provides general comments about using correlations between the index parameters and engineering soil properties contained in this appendix. The correlations are appropriate for preliminary engineering evaluations. For noncritical design problems - i.e., where the safety and economic consequences of failure are not important - use of the engineering property values derived from these correlations may be suitable for final engineering design, if the limitations of the property values are recognized. Many of these correlations received wide acceptance in the geotechnical engineering community. However, where the economic and safety consequences of the problem are important, a program of detailed exploration, sampling, and engineering testing is recommended for deriving engineering soil properties for design.

The correlations in this appendix have been organized under the following general headings:

1. Engineering Use Charts
2. General Correlations Applying to All Soil Types
3. Correlations for Granular Soils
4. Correlations for Cohesive Soils
5. Correlations for Field Index Tests
6. Evaluation of Freeze-Thaw Resistance
7. Evaluation of Swell Potential
8. Evaluation of Permeability

Figure A-0 will help locate the required correlation table or figure. The first column indicates engineering performance characteristics, and the top row lists soil property tests. The numbers in the boxes indicate the numbers of the various sections of the appendix and the corresponding tables and

Note: 1. The numbers in the table indicate the section of Appendix A where the particular correlation may be found.
2. X indicates that a relationship exists between an engineering performance characteristic and an index property test; however, no numerical correlation was identified.

ENGINEERING PERFORMANCE CHARACTERISTICS			CLASSIFICATION			LABORATORY ENGINEERING PROPERTY TESTS			
			UNIFIED	FAA	AASHTO	CONSOLID-ATION (C)	TRIAXIAL (T)	DIRECT SHEAR (DS) AND SIMPLE SHEAR (SS)	PERMEA-BILITY (P)
SUBSURFACE CONDITIONS		SOIL TYPES	1.3	1.3	1.3				
SUBSURFACE CONDITIONS		GROUNDWATER LEVELS							
MECHANICAL	IN-SITU STRESSES	DENSITY, γ	1.1 1.2						
MECHANICAL	IN-SITU STRESSES	COEFICIENT OF LATERAL EARTH EARTH PRESSURE, K_o					3.5 4.5.1	3.5	
MECHANICAL	IN-SITU STRESSES	OVERCONSOLIDATION RATIO, OCR				X			
MECHANICAL	SHEAR STRENGTH	DRAINED STRENGTH $\bar{c},\bar{\phi}$	1.2 3.6						
MECHANICAL	SHEAR STRENGTH	UNDRAINED STRENGTH, S_u	1.2					4.1.1 4.1.2	
MECHANICAL	COMPRESSIBILITY AND STIFFNESS	DRAINED DEFORMATION E, k (ELASTIC MODULUS, SETTLEMENT, CSR)	1.1 1.2				3.1 3.2		
MECHANICAL	COMPRESSIBILITY AND STIFFNESS	UNDRAINED DEFORMATION E_u, k (ELASTIC MODULUS, SETTLEMENT)	1.1 1.2				4.6.1 4.6.2 4.6.3	4.6	
MECHANICAL	COMPRESSIBILITY AND STIFFNESS	CONSOLIDATION INDICES C_c, C_R, C_s	1.1			X			
MECHANICAL	COMPRESSIBILITY AND STIFFNESS	SECONDARY COMPRESSION C				X			
MECHANICAL	COMPRESSIBILITY AND STIFFNESS	COEFICIENT OF CONSOLIDATION, C_v				X			
ENVIRONMENTAL		FREEZE-THAW RESISTANCE (SEE 6.1 & 6.2)	1.1 6.3						
ENVIRONMENTAL		SWELL POTENTIAL				X			
PERMEA-BILITY		PERMEABILITY	1.1 1.2 8.1			X	X		X

FIGURE A-0. INDEX TEST/ENGINEERING PROPERTY CORRELATIONS

Note: 1. The numbers in the table indicate the section of Appendix A where the particular correlation may be found.
2. X indicates that a relationship exists between an engineering performance characteristic and an index property test; however, no numerical correlation was identified.

ENGINEERING PERFORMANCE CHARACTERISTICS			LABORATORY INDEX PROPERTY TESTS								
			GRADATION	PHASE RELATIONS (γ, e, w, D_r)	PLASTICITY (ATTERBERG LIMITS) (AL)	UNCONFINED COMP. STRENGTH (q_u)	CALIFORNIA BEARING RATIO (CBR)	ORGANIC CONTENT	MINEROLOGY (CEC, X-RAY)	FROST HEAVE TESTS	SWELL TESTS
SUBSURFACE CONDITIONS		SOIL TYPES	X				1.2		X		
SUBSURFACE CONDITIONS		GROUNDWATER LEVELS									
MECHANICAL	IN-SITU STRESSES	DENSITY, γ						X			
MECHANICAL	IN-SITU STRESSES	COEFICIENT OF LATERAL EARTH EARTH PRESSURE, K_o			4.5.1 4.5.2						
MECHANICAL	IN-SITU STRESSES	OVERCONSOLIDATION RATIO, OCR			4.5.2						
MECHANICAL	SHEAR STRENGTH	DRAINED STRENGTH $\bar{c}, \bar{\phi}$	3.6.1	3.6	4.2 4.4				X		
MECHANICAL	SHEAR STRENGTH	UNDRAINED STRENGTH, S_u			4.1 4.3	X					
MECHANICAL	COMPRESSIBILITY AND STIFFNESS	DRAINED DEFORMATION E, k (ELASTIC MODULUS, SETTLEMENT, CSR)		3.3.1 3.3.2			3.4				
MECHANICAL	COMPRESSIBILITY AND STIFFNESS	UNDRAINED DEFORMATION E_u, k (ELASTIC MODULUS, SETTLEMENT)			4.6.1 4.6.2 4.6.3	X					
MECHANICAL	COMPRESSIBILITY AND STIFFNESS	CONSOLIDATION INDICES C_c, C_R, C_s			4.7.1			X	X		
MECHANICAL	COMPRESSIBILITY AND STIFFNESS	SECONDARY COMPRESSION C		4.9							
MECHANICAL	COMPRESSIBILITY AND STIFFNESS	COEFICIENT OF CONSOLIDATION, C_v			4.8						
ENVIRONMENTAL		FREEZE-THAW RESISTANCE (SEE 6.1 & 6.2)								6.3	
ENVIRONMENTAL		SWELL POTENTIAL	7.1		4.7.2 4.7.3 7.1 7.2				X		X
PERMEABILITY		PERMEABILITY	8.1 8.2 8.3 8.4	8.3	8.4						

FIGURE A-0. INDEX TEST/ENGINEERING PROPERTY CORRELATIONS (CONTINUED)

Note: 1. The numbers in the table indicate the section of Appendix A where the particular correlation may be found.
2. X indicates that a relationship exists between an engineering performance characteristic and an index property test; however, no numerical correlation was identified.

ENGINEERING PERFORMANCE CHARACTERISTICS			FIELD TESTS							
			STANDARD PENETRATION TEST (SPT)	CONE PENETRATION TEST (CPT)	VANE SHEAR TEST (VST)	PLATE BEARING TEST (PBT)	PIEZOMETERS	BOREHOLE PERM. AND PERC. TESTS	SEISMIC REFRACTION	ELECTRICAL RESISTIVITY
SUBSURFACE CONDITIONS		SOIL TYPES		5.6		1.1 1.2			2.1	2.2
SUBSURFACE CONDITIONS		GROUNDWATER LEVELS					X		X	X
MECHANICAL	IN-SITU STRESSES	DENSITY, γ	5.2	5.8					X	
MECHANICAL	IN-SITU STRESSES	COEFICIENT OF LATERAL EARTH EARTH PRESSURE, K_0								
MECHANICAL	IN-SITU STRESSES	OVERCONSOLIDATION RATIO, OCR								
MECHANICAL	SHEAR STRENGTH	DRAINED STRENGTH $\bar{c}, \bar{\phi}$	5.3	5.7 5.10						
MECHANICAL	SHEAR STRENGTH	UNDRAINED STRENGTH, S_u	5.4	5.10	4.1 5.5					
MECHANICAL	COMPRESSIBILITY AND STIFFNESS	DRAINED DEFORMATION E, k (ELASTIC MODULUS, SETTLEMENT, CSR)	5.1	5.11		2.3 3.4				
MECHANICAL	COMPRESSIBILITY AND STIFFNESS	UNDRAINED DEFORMATION E_u, k (ELASTIC MODULUS, SETTLEMENT)			4.6.3	2.3			X	
MECHANICAL	COMPRESSIBILITY AND STIFFNESS	CONSOLIDATION INDICES C_c, C_R, C_s								
MECHANICAL	COMPRESSIBILITY AND STIFFNESS	SECONDARY COMPRESSION C								
MECHANICAL	COMPRESSIBILITY AND STIFFNESS	COEFICIENT OF CONSOLIDATION, C_v								
ENVIRONMENTAL		FREEZE - THAW RESISTANCE (SEE 6.1 & 6.2)								
ENVIRONMENTAL		SWELL POTENTIAL								
PERMEABILITY		PERMEABILITY						X		

FIGURE A-0. INDEX TEST/ENGINEERING PROPERTY CORRELATIONS (CONTINUED) figures contained within this appendix. For example, to locate a correlation between undrained strength (S_u) and plasticity or Atterberg Limits, one should look in the appendix in section A4.1.1.

A1. ENGINEERING USE CHARTS

Engineering use charts provide descriptions of typical soil behavior and typical ranges of soil parameters for each soil group.

A1.1 Engineering Uses for Unified Soil Groups

Table A-1.1 presents qualitative evaluations of the uses of soils according to the Unified soil classification (USC) groups. In addition, some of the index and engineering properties are given. This table was developed from similar charts in the U. S. Army Corps of Engineers Technical Memorandum 3-357 "Characteristics of Soil Groups Pertaining to Roads and Airfields" and the 1978 AREA Manual for Railway Engineering, Chapter I, ("Roadbed").

A1.2 Typical Properties of Compacted Soils

Table A-1.2 lists index and engineering properties for soils by USC group. The properties are for soils compacted at the maximum standard or modified Proctor density (refer to ASTM Methods D698 and D1557), as indicated in the table notes.

A1.3 Soil Group Correlations

Table A-1.3 lists the FAA classification system and AASHTO classification system (AASHO System) soil groups that correspond to each of the USC soil groups. This table is useful for converting from USC to another system when the soils have been classified according to the Unified system but correlations with engineering performance have been developed based on another engineering classification system.

TABLE A-1.1. UNIFIED CLASSIFICATION SYSTEM SOIL GROUPS: CHARACTERISTICS AND USES

Major Divisions		Letter (1)	Name	Unit Dry Weight Lb per Cu Ft (2)	Field CBR	Subgrade Modulus k Lb per Cu In	Compressibility and Expansion	Potential Frost Action (3)	Drainage Characteristics (4)	Value as Filter Level (5)	Erosion on Exposed Slope (6)	Value of Subgrade (7)	Pumping Action (8)	Stability in Compacted Fills (9)	Compaction Characteristics (10)
COARSE GRAINED SOILS	GRAVEL AND GRAVELLY SOILS	GW	Well-graded gravels or gravel-sand mixtures, little or no fines	125-140	60-80	300 or more	Almost none	None to very slight	Excellent	Fair	None *	Excellent	None	Very good	Excellent; crawler-type tractor, rubber-tired roller, steel-wheeled roller
		GP	Poorly graded gravels or gravel-sand mixtures, little or no fines	110-130	25-60	300 or more	Almost none	None to very slight	Excellent	Fair to poor	None *	Excellent	None	Reasonably good	Good; crawler-type tractor, rubber-tired roller, steel-wheeled roller
		GM d	Silty gravels, gravel-sand-silt mixtures	130-145	40-80	300 or more	Very slight	Slight to medium	Fair to poor	Very poor	None to slight	Good	None	Reasonably good	Good with close moisture control; rubber-tired roller, sheepfoot roller
		GM u		120-140	20-40	200 to 300	Slight	Slight to medium	Poor to practically impervious						
		GC	Clayey gravels, gravel-sand-clay mixtures	120-140	20-40	200 to 300	Slight	Slight to medium	Poor to practically impervious	Not to be used	None to slight	Good	Slight	Fair	Excellent; rubber-tired roller, sheepfoot roller
	SAND AND SANDY SOILS	SW	Well-graded sands or gravelly sands, little or no fines	110-130	20-40	200 to 300	Almost none	None to very slight	Excellent	Excellent	Slight to high with decreasing gravel content	Excellent	None	Very good	Excellent; crawler-type tractor, rubber-tired roller
		SP	Poorly grades sands or gravelly sands, little or no fines	100-120	10-25	200 to 300	Almost none	None to very slight	Excellent	Fair to poor	High	Good	None	Reasonably good with flat slopes	Good; crawler-type tractor, rubber-tired roller
		SM d	Silty sands, sand-silt mixture	120-135	20-40	200 to 300	Very slight	Slight to high	Fair to poor	Very poor	High	Poor	None to slight	Fair	Good with close moisture control; rubber-tired roller, sheepfoot roller
		SM u		105-130	10-20	200 to 300	Slight to medium	Slight to high	Poor to practically impervious						
		SC	Clayey sands, sand-clay mixtures	105-130	10-20	200 to 300	Slight to medium	Slight to high	Poor to practically impervious	Not to be used	Slight	Poor	Slight	Fair	Excellent; rubber-tired roller, sheepfoot roller
FINE GRAINED SOILS	SILTS AND CLAYS LL < 50	ML	Inorganic silts and very fine sands, rock flour, silty or clayey fine sands or clayey silts with slight plasticity	100-125	5-15	100 to 200	Slight to medium	Medium to very high	Fair to poor	Not to be used	Very high	Poor	Slight to bad	Poor	Poor to good with close control of moisture; rubber-tired roller, sheepfoot roller
		CL	Inorganic clays of low to medium plasticity, gravelly clays, sandy clays, silty clays, lean clays	100-125	5-15	100 to 200	Medium	Medium to high	Practically impervious	Not to be used	None to slight	Bad	Bad	Reasonable	Fair to good; rubber-tired roller, sheepfoot roller
		OL	Organic silts and organic silt-clays of low plasticity	90-105	4-8	100 to 200	Medium to high	Medium to high	Poor	Not to be used	Variable	Bad	Very bad	Not to be used	Poor to very poor
	SILTS AND CLAYS LL > 50	MH	Inorganic silts, micaceous or diatomaceous fine sandy or silty soils, plastic silts	80-100	4-8	100 to 200	High	Medium to very high	Fair to poor	Not to be used	None to slight	Bad	Very bad	Poor	Poor to very poor; sheepfoot roller
		CH	Inorganic clays of high plasticity, fat clays	90-110	3-5	50 to 100	High	Medium	Practically impervious	Not to be used	None	Bad	Very bad	Fair with flat slopes	Fair to poor; sheepfoot roller
		OH	Organic clays of medium to high plasticity, organic silts	80-105	3-5	50 to 100	High	Medium	Practically impervious	Not to be used	Variable	Bad	Very bad	Not to be used	Poor to very poor
HIGHLY ORGANIC SOILS		Pt	Peat and other highly organic soils	—	—	—	Very high	Slight	Fair to poor	Not to be used	Not applicable	Remove completely	Very bad	Not to be used	Compaction not possible

NOTES:

(1) Division of GM and SM groups into subdivisions of d and u are for roads and airfields only; subdivision is on basis of Atterberg limits; suffix d (e.g. GMd) will be used when the liquid limit is 28 or less and the plasticity index is 6 or less; the suffix u will be used when the liquid limit is greater than 28.

(2) Unit dry weights are for compacted soil at optimum moisture content for modified AASHO compactive effort.

(3) These soils are susceptible to frost as indicated under conditions favorable to frost action described in the text.

(4) Ability of soil to drain water by gravity. Drainage ability decreases with decreasing average grain size.

(5) Value of soil as filter backfill around subdrain pipes to prevent clogging with fines, and as filter layer to prevent migrations of fines from below.

(6) Ability of natural soil to resist erosion on an exposed slope. Soils marked * may be used to protect eroding slopes of other materials.

(7) Value as stable subgrade for roadbed, when protected by suitable ballast and subballast material. Good soils may be used to protect poorer soils in subgrade.

(8) Tendency of soil to pump up and foul ballast under traffic.

(9) Stability of soil against bulging and subsidence when used in a rolled fill. Cross-check with column (6) to forecast tendency to erode.

(10) The equipment listed will usually produce the required densities with a reasonable number of passes when moisture conditions and thickness of lift are properly controlled. In some instances, several types of equipment are listed, because variable soil characteristics within a given soil group may require different equipment. In some instances, a combination of two types may be necessary.

REFERENCES: U.S. Army Corps of Engineers, 1953, "The Unified Soil Classification System", Technical Memo. 3-357, Appendix B, Table B-1.

AREA, (1976) Manual for Railway Engineering, p 1-1-32/33.

TABLE A-1.2. TYPICAL PROPERTIES OF COMPACTED MATERIALS

Group symbol	Soil type	Range of maximum dry unit weight, pcf	Range of optimum moisture, percent	Typical value of compression: At 1.4 tsf (20 psi) percent of original height	Typical value of compression: At 3.6 tsf (50 psi) percent of original height	Typical strength characteristics: Cohesion (as compacted) psf	Typical strength characteristics: Cohesion (saturated) psf	Typical strength characteristics: φ(Effective stress envelope) degrees	Typical strength characteristics: Tan φ	Typical coefficient of permeability ft/min.	Range of CBR values	Range of subgrade modulus k lb/cu in.
GW	Well graded clean gravels, gravel-sand mixtures.	125 - 135	11 - 8	0.3	0.6	0	0	>38	>0.79	5×10^{-2}	40 - 80	300 - 500
GP	Poorly graded clean gravels, gravel-sand mix.	115 - 125	14 - 11	0.4	0.9	0	0	>37	>0.74	10^{-1}	30 - 60	250 - 400
GM	Silty gravels, poorly graded gravel-sand-silt.	120 - 135	12 - 8	0.5	1.1	....	....	>34	>0.67	$>10^{-6}$	20 - 60	100 - 400
GC	Clayey gravels, poorly graded gravel-sand-clay.	115 - 130	14 - 9	0.7	1.6	....	....	>31	>0.60	$>10^{-7}$	20 - 40	100 - 300
SW	Well graded clean sands, gravelly sands.	110 - 130	16 - 9	0.6	1.2	0	0	38	0.79	$>10^{-3}$	20 - 40	200 - 300
SP	Poorly graded clean sands, sand-gravel mix.	100 - 120	21 - 12	0.8	1.4	0	0	37	0.74	$>10^{-3}$	10 - 40	200 - 300
SM	Silty sands, poorly graded sand-silt mix.	110 - 125	16 - 11	0.8	1.6	1050	420	34	0.67	5×10^{-5}	10 - 40	100 - 300
SM-SC	Sand-silt clay mix with slightly plastic fines.	110 - 130	15 - 11	0.8	1.4	1050	300	33	0.66	2×10^{-6}	....	
SC	Clayey sands, poorly graded sand-clay mix.	105 - 125	19 - 11	1.1	2.2	1550	230	31	0.60	5×10^{-7}	5 - 20	100 - 300
ML	Inorganic silts and clayey silts .	95 - 120	24 - 12	0.9	1.7	1400	190	32	0.62	10^{-5}	15 or less	100 - 200
ML-CL	Mixture of inorganic silt and clay	100 - 120	22 - 12	1.0	2.2	1350	460	32	0.62	5×10^{-7}	....	
CL	Inorganic clays of low to med. plasticity.	95 - 120	24 - 12	1.3	2.5	1800	270	28	0.54	10^{-7}	15 or less	50 - 200
OL	Organic silts and silt-clays, low plasticity.	80 - 100	33 - 21	....	....	....	....	....	.....	....	5 or less	50 - 100
MH	Inorganic clayey silts, elastic silts.	70 - 95	40 - 24	2.0	3.8	1500	420	25	0.47	5×10^{-7}	10 or less	50 - 100
CH	Inorganic clays of high plasticity	75 - 105	36 - 19	2.6	3.9	2150	230	19	0.35	10^{-7}	15 or less	50 - 150
OH	Organic clays and silty clays . . .	65 - 100	45 - 21	....	....	....	....	....	.....	....	5 or less	25 - 100

Notes:
1. All properties are for condition of "standard Proctor" maximum density, except values of k and CBR which are for "modified Proctor" maximum density.
2. Typical strength characteristics are for effective strength envelopes and are obtained from USBR data.
3. Compression values are for vertical loading with complete lateral confinement.
4. (>) indicates that typical property is greater than the value shown.
(.......) indicates insufficient data available for an estimate.

Reproduced from Soil Mechanics, Foundations, and Earth Structures, p. 7-9-2, U.S. NAVFAC Design Manual DM-7. Year of first publication: 1961.

TABLE A-1.3. COMPARABLE SOIL GROUPS FOR UNIFIED SOIL CLASSIFICATION GROUPS

Soil Group in Unified System	Comparable Soil Groups in FAA System			Comparable Soil Groups in AASHO System		
	Most Probable	Possible	Possible but Improbable	Most Probable	Possible	Possible but Improbable
GW	E-1	—	—	A-1-a	—	A-2-4, A-2-5, A-2-6, A-2-7
GP	E-1	—	—	A-1-a	A-1-b	A-3, A-2-4, A-2-5, A-2-6, A-2-7
GM	E-2, E-4, E-5	—	E-1, E-6, E-7, E-8, E-9, E-10, E-11, E-12	A-1-b, A-2-4, A-2-5, A-2-7	A-2-6	A-4, A-5, A-6, A-7-5, A-7-6, A-1-a
GC	E-5	E-4	E-6, E-7, E-8, E-10 E-11, E-12	A-2-6, A-2-7	A-2-4, A-6	A-4, A-7-6, A-7-5
SW	E-1	—	—	A-1-b	A-1-a	A-3, A-2-4, A-2-5, A-2-6, A-2-7
SP	E-1, E-3	E-2	—	A-3, A-1-b	A-1-a	A-2-4, A-2-5, A-2-6, A-2-7
SM	E-2, E-3, E-4, E-5	—	E-1, E-6, E-7, E-8, E-9, E-10, E-11, E-12	A-1-b, A-2-4, A-2-5, A-2-7	A-2-6, A-4, A-5	A-6, A-7-5, A-7-6, A-1-a
SC	E-4, E-5	—	E-6, E-7, E-8, E-10, E-11, E-12	A-2-6, A-2-7	A-2-4, A-6, A-4, A-7-6	A-7-5
ML	E-6, E-7	E-9	E-1, E-2, E-3, E-5	A-4, A-5	A-6, A-7-5,	—
CL	E-7	E-6, E-8	E-4, E-5	A-6, A-7-6	A-4	—
OL	E-6, E-7	E-9	E-1, E-2, E-3, E-5	A-4, A-5	A-6, A-7-5, A-7-6	—
MH	E-8, E-9, E-10, E-11, E-12	—	—	A-7-5, A-5	—	A-7-6
CH	E-8, E-10, E-11, E-12	—	—	A-7-6	A-7-5	—
OH	E-8, E-9, E-10, E-11, E-12	—	—	A-7-5, A-5	—	A-7-6
Pt	E-13	—	—	—	—	—

Reproduced from "A Review of Engineering Soil Classification Systems," p. 17, by T.K. Liu from Highway Research Record, Number 156. Year of first publication: 1967.

A2. GENERAL CORRELATIONS FOR ALL SOIL TYPES

The following correlations are for both cohesive and cohesionless soils as well as rocks.

A2.1 Typical Seismic Velocities of Earth Materials

The velocities listed in the table below are typical compression wave (p-wave) velocities which are observed in seismic geophysical explorations.

TABLE A-2.1. TYPICAL SEISMIC VELOCITIES OF EARTH MATERIALS

Material	Velocity (ft/sec)
Dry silt, sand, loose gravel, loam, loose rock, talus, and moist fine-grained topsoil	600-2500
Compact till; indurated clays; gravel below water table,* compact clayey gravel, cemented sand, and sand-clay	2500-7500
Rock, weathered, fractured, or partly decomposed	2000-10,000
Shale, sound	2500-11,000
Sandstone, sound	5000-14,000
Limestone, chalk, sound	6000-20,000
Igneous rock, sound	12,000- 20,000
Metamorphic rock, sound	10,000- 16,000

* Velocity of sound in water is about 4700 ft/sec and all fully saturated materials should have velocities equal to or exceeding this value.

From Foundation Engineering, second edition, p. 120, by R. Peck et al. by permission of John Wiley and Sons. Year of first publication: 1974.

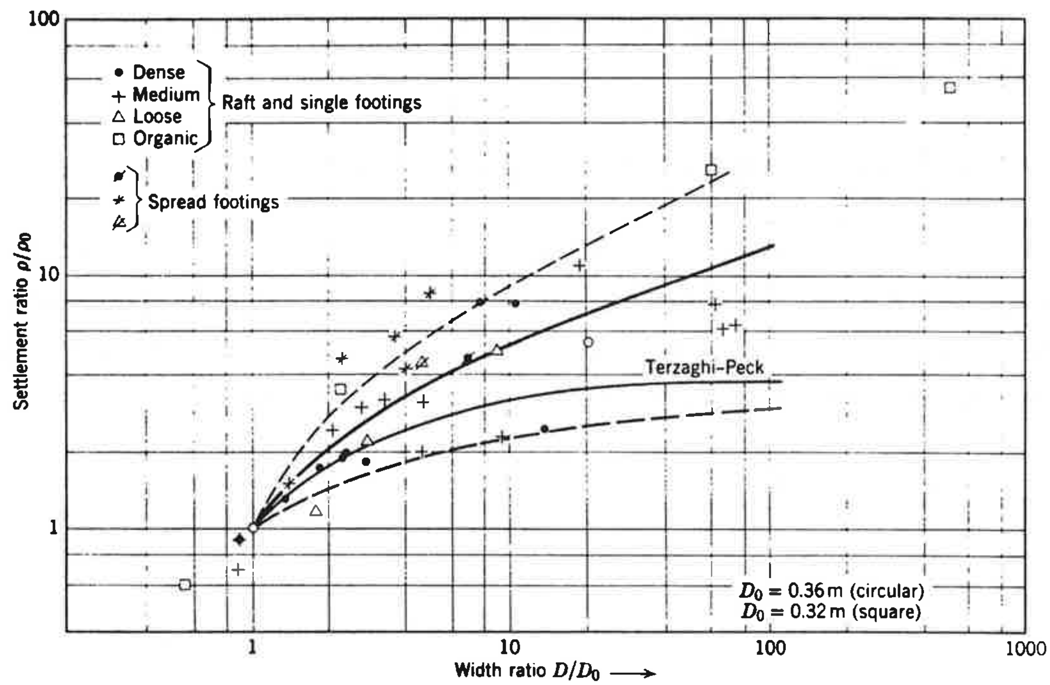
A2.2 Representative Values of Electrical Resistivity

The resistivities listed in the table below are representative of the values that are measured by electrical resistivity geophysical surveys.

TABLE A-2.2. REPRESENTATIVE VALUES OF ELECTRICAL RESISTIVITY

Material	Resistivity (ohm-cm)
Clay and saturated silt	0-10,000
Sandy clay and wet silty sand	10,000-25,000
Clayey sand and saturated sand	25,000-50,000
Sand	50,000-150,000
Gravel	150,000-500,000
Weathered rock	100,000-200,000
Sound rock	150,000-4,000,000

From Foundation Engineering, second edition, p. 120, by R. Peck et al. by permission of John Wiley & Sons, Inc. Year of first publication: 1974.



D_0 = reference diameter of plate or footing

D = diameter of footing or raft

ρ = settlement of plate of width D

ρ_0 = settlement of footing or raft of width D_0

Reproduced from Soil Mechanics, p. 220, by T. W. Lambe and R. V. Whitman, by permission of John Wiley & Sons, Inc. Year of first publication: 1969.

FIGURE A-2.3. COMPARISON BETWEEN SETTLEMENT AND DIMENSION OF LOADED AREA AS DERIVED FROM COLLECTED CASE RECORDS (FROM BJERRUM AND EGGESTAD, 1963)

A3. CORRELATIONS FOR GRANULAR SOILS

The following correlations apply only to granular soils; i.e., sands and gravels.

A3.1 Typical Values of Young's Modulus for Initial Loading

The following table provides typical values of initial secant Young's modulus for drained triaxial loading of dry sands. The secant modulus represents the straight line that connects the zero stress-strain condition to the stress-strain state at one-half of the peak deviator stress.

TABLE A-3.1. TYPICAL VALUES OF YOUNG'S MODULUS FOR INITIAL LOADING

Description of sand	Loose		Dense	
	MPa	psi	MPa	psi
Angular, breakable particles	14	2000	35	5000
Hard, rounded particles	56	8000	105	15000

From Soil Mechanics, p. 159, by T. W. Lambe & R. V. Whitman. Year of first publication: 1969.

A3.2 Typical Values of Young's Modulus for Repeated Loading

The table below provides values of Young's modulus for dry sands for repeated or cyclic triaxial loading of dry sands.

TABLE A-3.2. TYPICAL VALUES OF YOUNG'S MODULUS FOR REPEATED LOADING

Soil (1 atm confining pressure)	Young's Modulus (psi)	
	Loose	Dense
Screened crushed quartz, fine angular	17,000	30,000
Screened Ottawa sand, fine rounded	26,000	45,000
Ottawa Standard sand, medium rounded	30,000	52,000
Screened sand, medium subangular	20,000	35,000
Screened crushed quartz,medium,angular	18,000	27,000
Well graded sand, coarse subangular	15,000	28,000

From "An Investigation of Stress-Strain and Strength Characteristics of Cohesionless Soils by Triaxial Compression Tests", p. 35, by L. S. Chen. Year of first publication: 1948.

A3.3 Shear Wave Velocity at Low Strain Levels

The data on low strain moduli of soils have been derived from resonant column shear wave laboratory tests on dry sands. The strain level for which these data are shown is 10^{-3} radians. The variation with void ratio and confining stress are shown in Figure A-3.3.1 and Figure A-3.3.2 for round and for angular sands.

A3.4 Young's Modulus, Coefficient of Subgrade Reaction, and California Bearing Ratio

In 1957, Nascimento developed relationships among Young's modulus (E), coefficient of subgrade reaction (K_s), and California bearing ratio (CBR) based on plate loading tests on various granular materials. The relation between Young's Modulus and CBR is shown in Figure A-3.4. The relation between K_s and CBR are as follows:

For a 0.75-m diameter plate:

$$K_s = \frac{CBR}{8} \quad \text{for soft materials (CBR} < 15)$$

$$K_s = \frac{CBR}{4} \quad \text{for hard materials (CBR} > 80)$$

A3.5 Relation Between Lateral Earth Pressure at Rest and Friction Angle

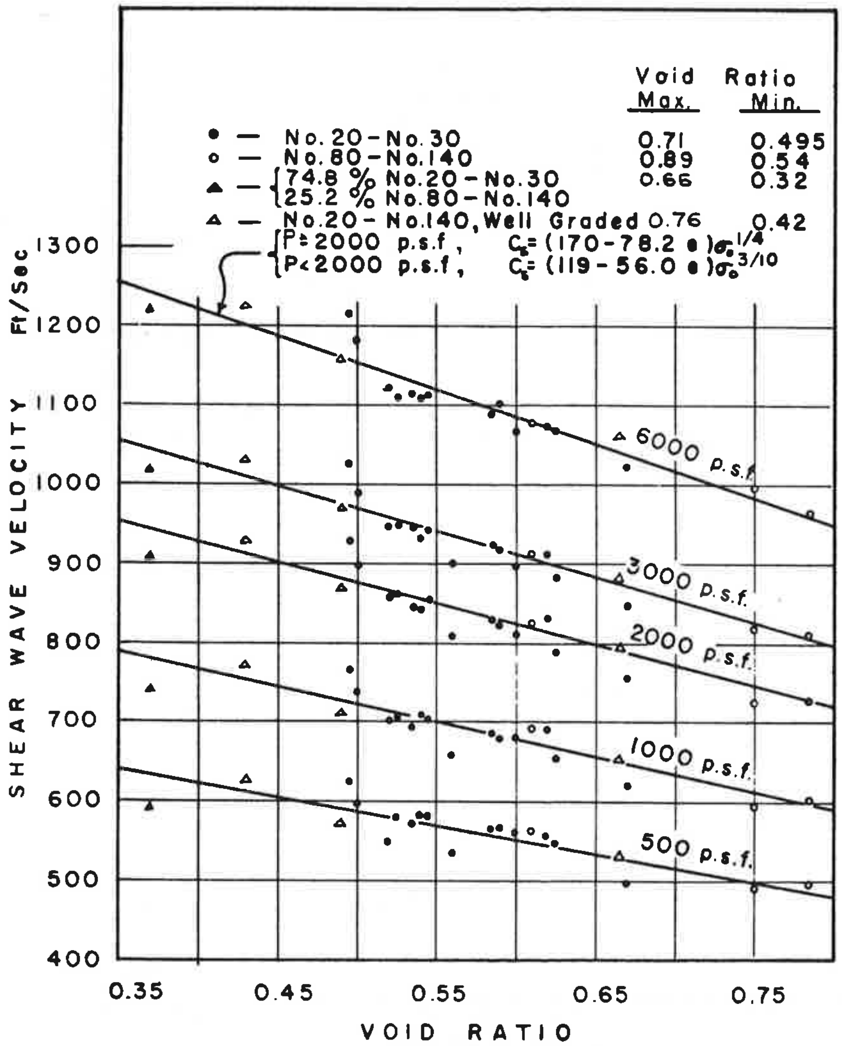
Based on experimental data, the relation between the lateral earth pressure coefficient at rest (K_o) and the drained, effective friction angle ($\bar{\phi}$) can be expressed as:

$$K_o = \bar{\sigma}_{ho} / \bar{\sigma}_{vo} = 1 - \sin\bar{\phi}$$

$\bar{\sigma}_{ho}$ = horizontal effective stress

$\bar{\sigma}_{vo}$ = vertical effective stress

Ladd states that the relation $K_o = 0.40 \pm 0.05$ appears to be as reliable as the former relation. Some data are shown in Figure A-3.5 below.



The lines on the figure that fit these data are represented by the following equations for e<0.80:

$$v_s = [170 - (78.2)e]\,(\bar{\sigma}_o)^{0.25} \qquad \begin{cases} v_s \text{ in feet/second} \\ \bar{\sigma}_o \text{ in pounds/ft}^2 \end{cases}$$

$$G = \frac{2630\,(2.17 - e)^2}{1 + e}(\bar{\sigma}_o)^{0.5} \qquad G \text{ and } \bar{\sigma}_o \text{ in pounds/in}^2$$

v_s = shear-wave velocity

G = shear modulus = $\rho\, v_s^2$

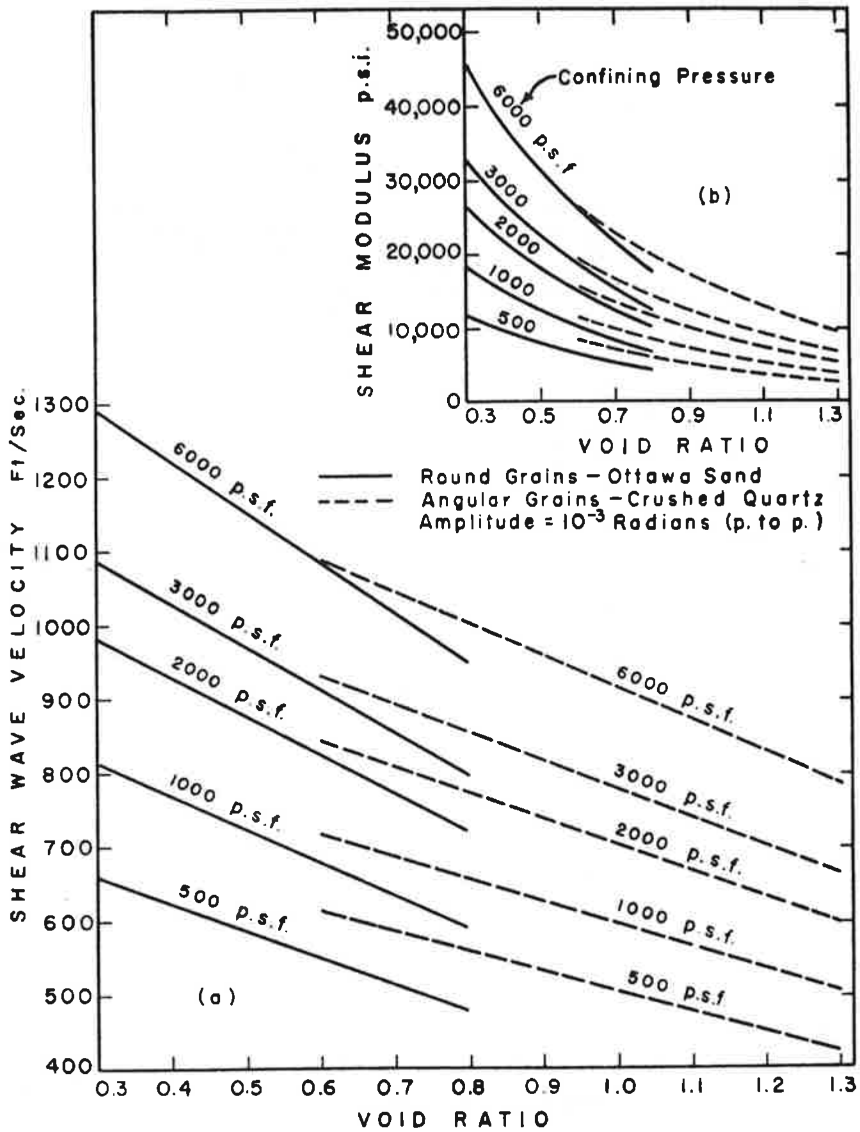
ρ = mass density

e = void ratio

$\bar{\sigma}_o$ = mean effective confining stress

Reproduced from "Elastic Wave Velocities in Granular Soils", p. 59, by B. Hardin & F. Richart by permission of A.S.C.E. Year of first publication: 1963.

FIGURE A-3.3.1. VARIATION OF SHEAR WAVE VELOCITY WITH VOID RATIO FOR VARIOUS CONFINING PRESSURES, GRAIN SIZES, AND GRADATIONS IN DRY OTTAWA SAND



The dashed lines for angular grained sands are represented by the following equations:

$$v_s = [159 - 53.5e](\bar{\sigma}_o)^{0.25}$$

$$G = \frac{1230[2.97 - e]^2}{(1 + e)} (\bar{\sigma}_o)^{0.5}$$

$\left\{\begin{array}{l} v_s \text{ in feet/ second} \\ \bar{\sigma}_o \text{ in pounds/ft}^2 \end{array}\right.$

G and $\bar{\sigma}$ in pounds/in^2

where-

v_s = shear wave velocity

G = shear modulus = ρv_s^2

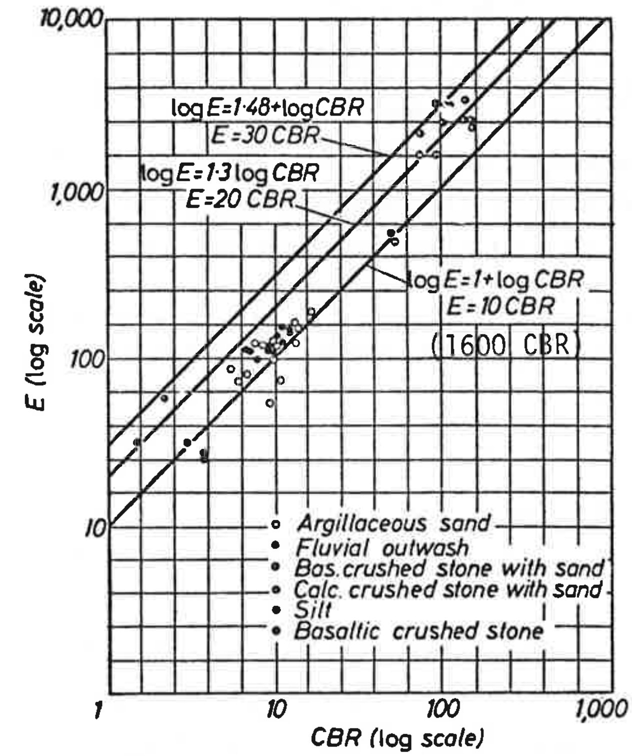
$\bar{\sigma}_o$ = mean effective confining stress

ρ = mass density

e = void ratio

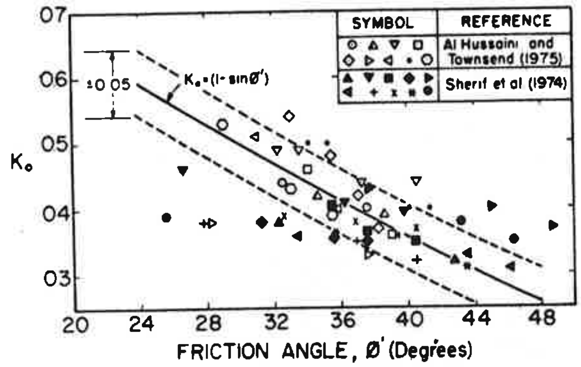
Reproduced from "Elastic Wave Velocities in Granular Soils", p.61, by B. Hardin and F. Richart by permission of A.S.C.E. Year of first publication: 1963.

FIGURE A-3.3.2. VARIATION OF SHEAR WAVE VELOCITY AND SHEAR MODULUS WITH VOID RATIO AND CONFINING PRESSURE FOR DRY ROUND AND ANGULAR GRAINED SANDS



Reproduced from "Relation Between CBR and Modulus of Strength," p. 167, by V. Nascimento and A. Simoes by permission of the Institution of Civil Engineers. Year of first publication: 1957.

FIGURE A-3.4. RELATIONSHIP BETWEEN CBR AND E,LOGARITHMIC SCALES



Reproduced from "Stress-Deformation and Strength Characteristics," p. 432, by C.C. Ladd et al. by permission of the Japanese Society of Soil Mechanics and Foundation Engineering. Year of first publication: 1977.

FIGURE A-3.5. RELATIONSHIP BETWEEN COEFFICIENT OF LATERAL EARTH PRESSURE AT REST AND FRICTION ANGLE FOR NORMALLY CONSOLIDATED SANDS

3.6 Friction Angle of Granular Soils

3.6.1 Summary of Friction Angle Data for Use in Preliminary Design

Table A-3.6.1 provides ranges of friction angles for various groups of granular soils. Within each range, use lower values if particles are well rounded or particles are soft and flaky (micaceous); use higher values for hard, angular particles.

TABLE A-3.6.1. SUMMARY OF FRICTION ANGLE DATA FOR USE IN PRELIMINARY DESIGN

	Friction Angles (degrees)		
	At Peak Strength		At Ultimate Strength (large strain)
Classification	medium dense $\bar{\phi}$	dense $\bar{\phi}$	$\bar{\phi}_{cv}$
Silt (non-plastic)	28 to 32	30 to 34	26 to 30
Uniform fine to medium sand	30 to 34	32 to 36	26 to 30
Well-graded sand	34 to 40	38 to 46	30 to 34
Sand and Gravel	36 to 42	40 to 48	32 to 36

Adapted from Basic Soils Engineering by I.K. Hough. Year of first publication: 1957.

A3.6.2 Angle of Internal Friction Versus Density

Figure A-3.6.2 shows the relation among effective stress angle of internal friction ($\bar{\Phi}$),density, and relative density. Typical ranges are shown for the Unified soil groups. This correlation is satisfactory for cohesionless soils only (cohesion intercept, c=0).

A3.6.3 Friction Angle and Initial Porosity

Figure A-3.6.3 illustrates the relations between friction angle and porosity for a variety of materials.

A3.6.4 Friction Angle versus Relative Density

Relative density is a parameter that can be used to characterize the in-place physical state of cohesionless soils. The parameter describes the in-place physical state within the continuous range of states from loosest possible to densest possible in terms of void ratio. The expression for relative density, D_r, is:

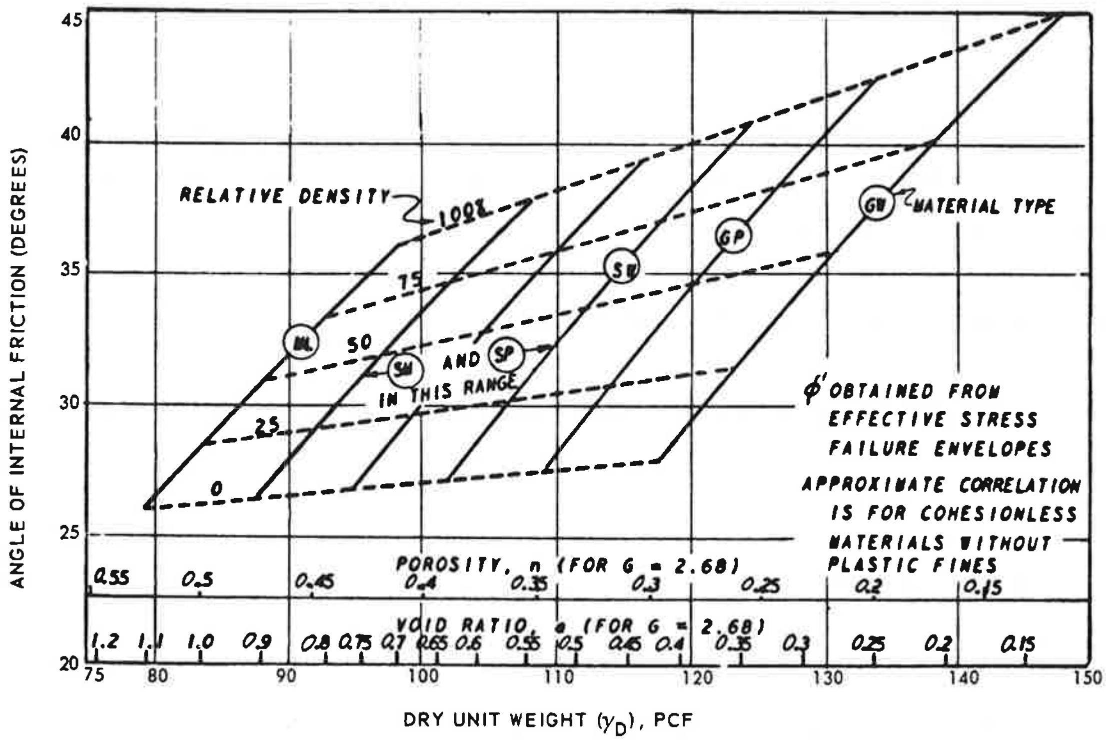
$$D_r = \frac{(e_{max} - e)}{(e_{max} - e_{min})} \times 100, \text{(in percent)}$$

where e_{min} = minimum void ratio (maximum density)

e_{max} = maximum void ratio (minimum density)

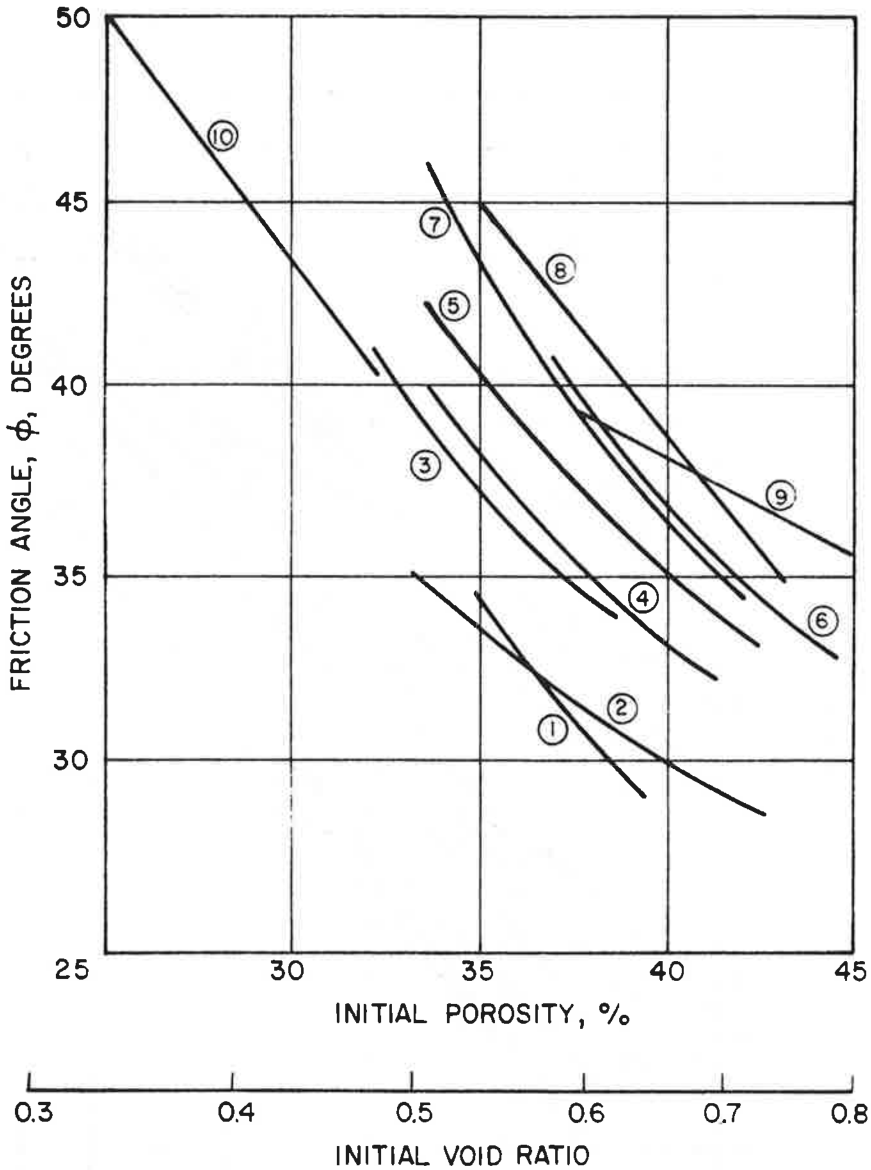
e = in-place void ratio

The extraordinary scatter shown by the relations plotted in Figure A-3.6.4 is an indication of the difficulties in using relative density as a measure of the physical state of soils.



Reproduced from Soil Mechanics, Foundations, and Earth Structures, p. 7-3-17, U.S. NAVFAC Design Manual DM-7. Year of first publication: 1961.

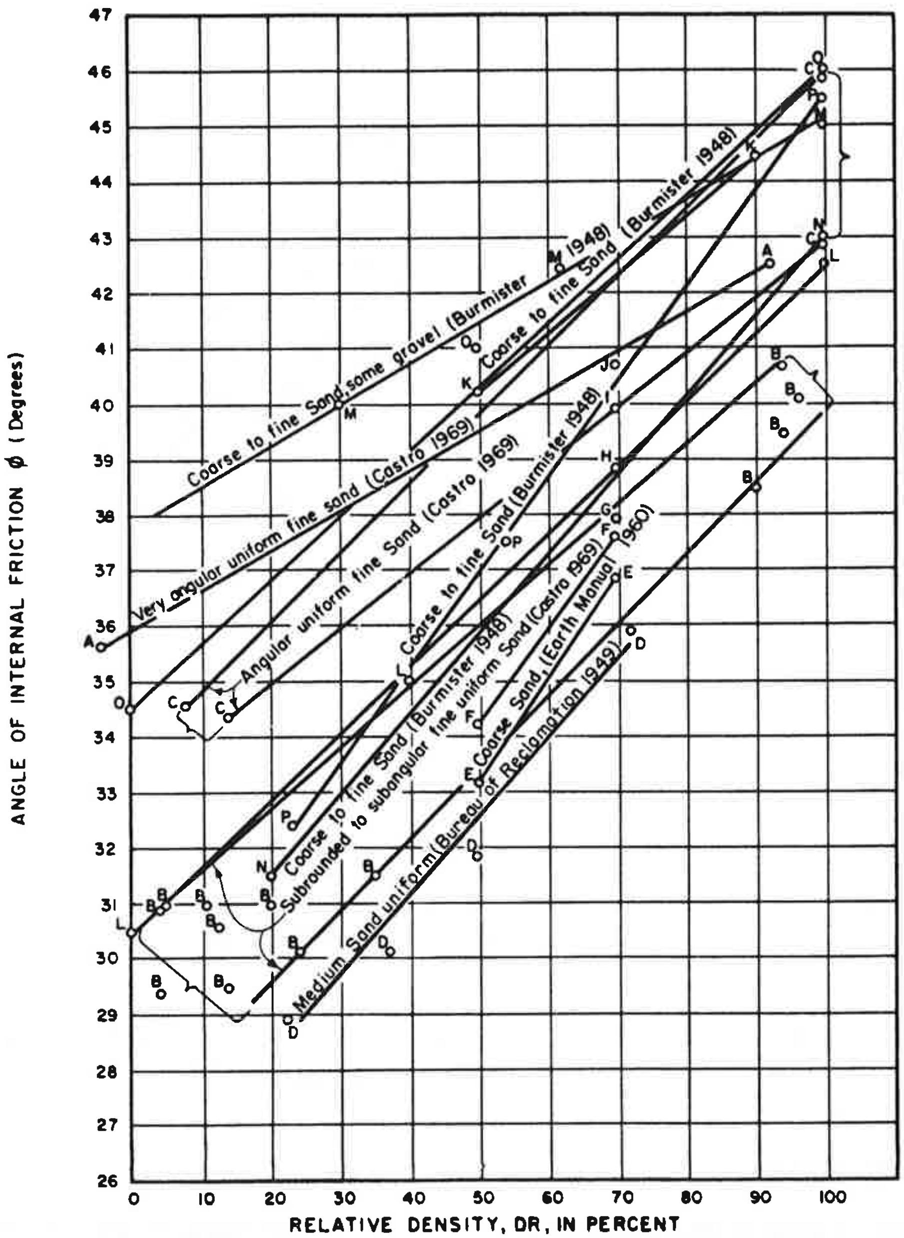
FIGURE A-3.6.2. ANGLE OF INTERNAL FRICTION VERSUS DENSITY (FOR COARSE-GRAINED SOILS)



Reference Key to Curves:

① Ottawa Sand (Lambe & Whitman, 1969)
② Fine Sand (Lambe & Whitman, 1969)
③ Sandy Gravel (Lambe & Whitman, 1969)
④ Medium Fine Sand (Rave, 1962)
⑤ Cornforth (1964) Triaxial
⑥ River Sand (Lambe & Whitman, 1969)
⑦ Cornforth (1964) Plane Strain
⑧ Pebbles (Lambe & Whitman, 1969)
⑨ River Sand (Lambe & Whitman, 1969)
⑩ Gravel (Lambe & Whitman, 1969)

FIGURE A-3.6.3. FRICTION ANGLE VERSUS INITIAL POROSITY FOR SEVERAL GRANULAR SOILS



Reproduced from Foundation Engineering Handbook, p. 263, by H.F. Winterkorn and H-Y. Fang by permission of Litton Educational Publishing, Inc. Year of first publication: 1975.

FIGURE A-3.6.4. RELATIVE DENSITY VERSUS FRICTION ANGLE FOR COHESIONLESS SOILS

A4. CORRELATIONS FOR COHESIVE SOILS

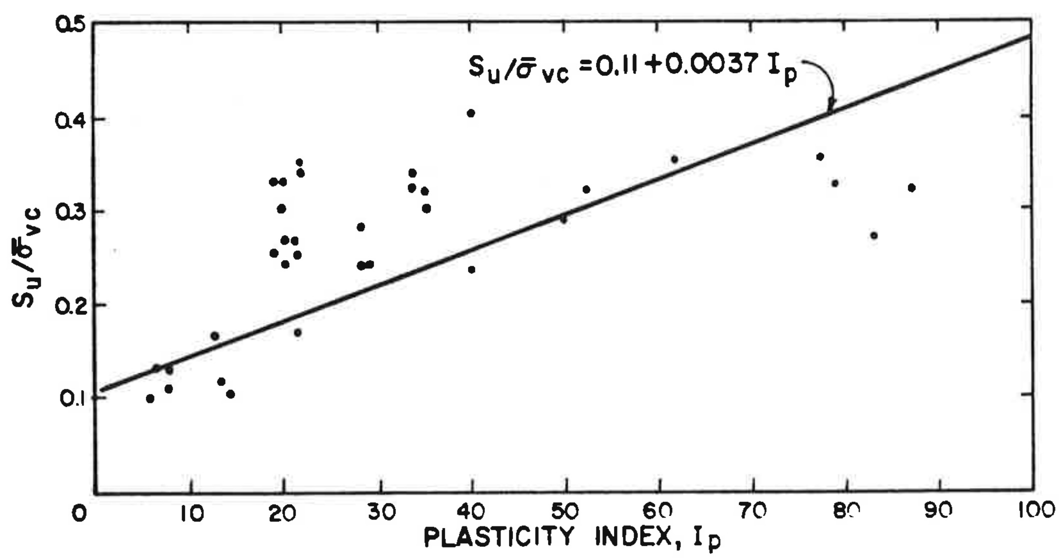
The following correlations apply to cohesive soils only, i.e., clays and plastic silts.

A4.1 Undrained Strength Ratio

The undrained strength ratio is defined as $S_u/\bar{\sigma}_{vc}$ where S_u represents the undrained shear strength and $\bar{\sigma}_{vc}$ represents the in situ vertical effective stress.

A4.1.1 Relation Between Field Vane Strength Ratio and Plasticity Index

Skempton and Henkel, in their 1953 paper on post-glacial clays, presented an empirical relation between undrained strength ratio, ($S_u/\bar{\sigma}_{vc}$) and the plasticity index (PI) for normally consolidated clays based upon field vane test data. The correlation was reviewed in 1973 by Sridharan and Rao in "The Relationship Between Undrained Strength and Plasticity Index," and is compared with published data points in Figure A-4.1.1. This correlation should be used only for a preliminary index measure of undrained strength.



Adapted from "The relationship between undrained strength and plasticity index" by Sridharan & Rao, 1973. Geotechnical Engineering, Vol. 4, p. 45.

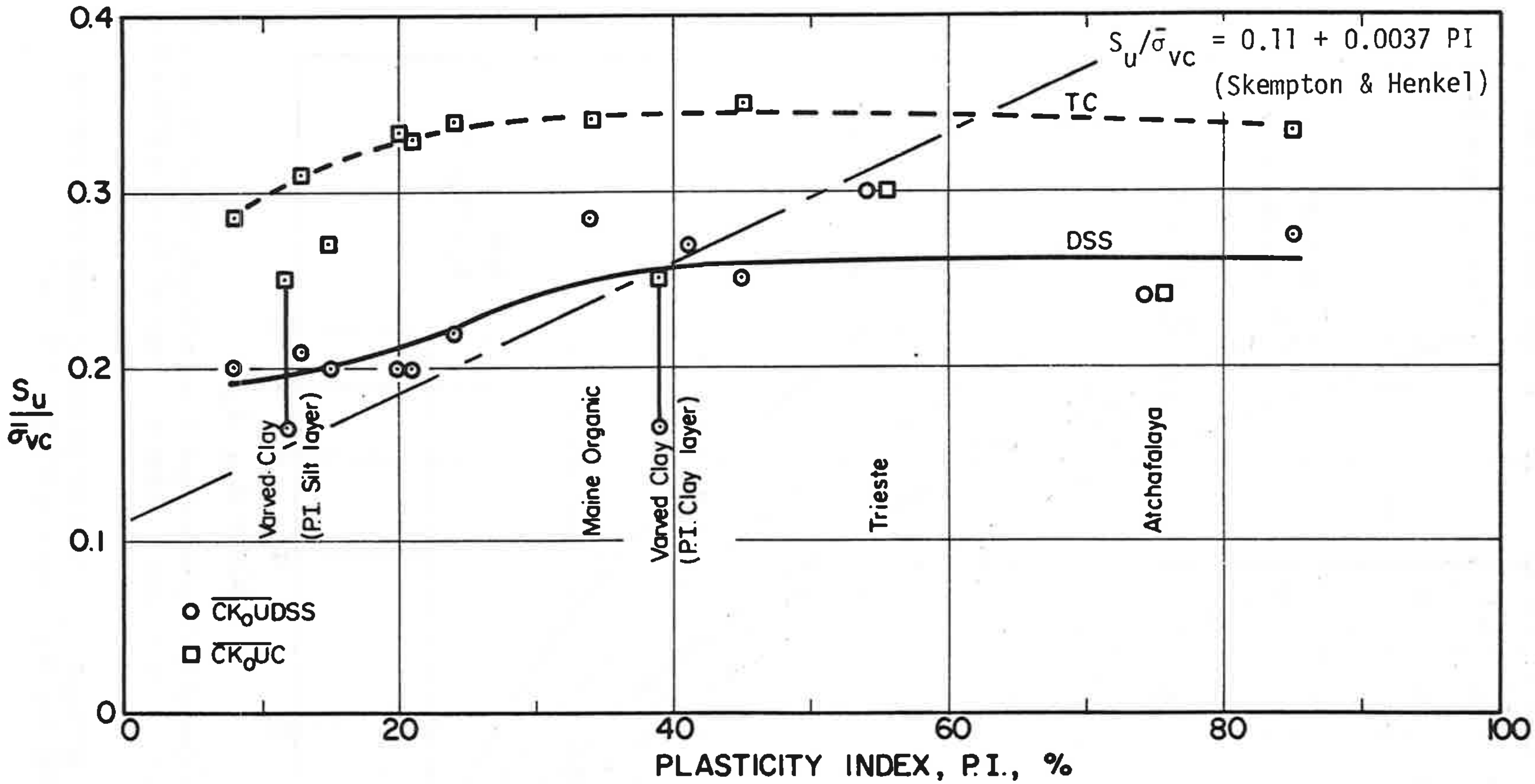
FIGURE A-4.1.1. RELATIONSHIP BETWEEN $S_u/\bar{\sigma}_{vc}$ AND PLASTICITY INDEX FOR FIELD VANE TESTS

A4.1.2 SHANSEP Correlations

SHANSEP refers to an approach for determining the strength of cohesive soils described by C. C. Ladd and R. Foote in the 1974 paper, "A New Design Procedure for Stability of Soft Clays." SHANSEP is an abbreviation for Stress History and Normalized Soil Engineering Properties.

In order to use the correlations developed by this approach, it is necessary to develop the stress history at the site. This means that the existing vertical effective stress ($\bar{\sigma}_{vc}$) and the maximum past effective vertical stress ($\bar{\sigma}_{vm}$) must be determined. These are normally developed from unit weight and piezometric data to calculate $\bar{\sigma}_{vc}$ and consolidation test data to determine $\bar{\sigma}_{vm}$. Atterberg Limits are used to characterize the soils.

Correlations to determine shear strength are shown in Figures A-4.1.2a and b. Figure A-4.1.2a shows the relation between strength ratio and plasticity index based on laboratory triaxial and Geonor direct-simple shear tests on relatively undisturbed tube samples. Figure A-4.1.2b shows the relation between strength ratio and overconsolidation ratio (OCR) for various clays. To use these relations for calculating the strength ratio of other clays, select one of the given relations or interpolate based on plasticity limits, as shown on the table within the figure.



S_u = Undrained Shear Strength

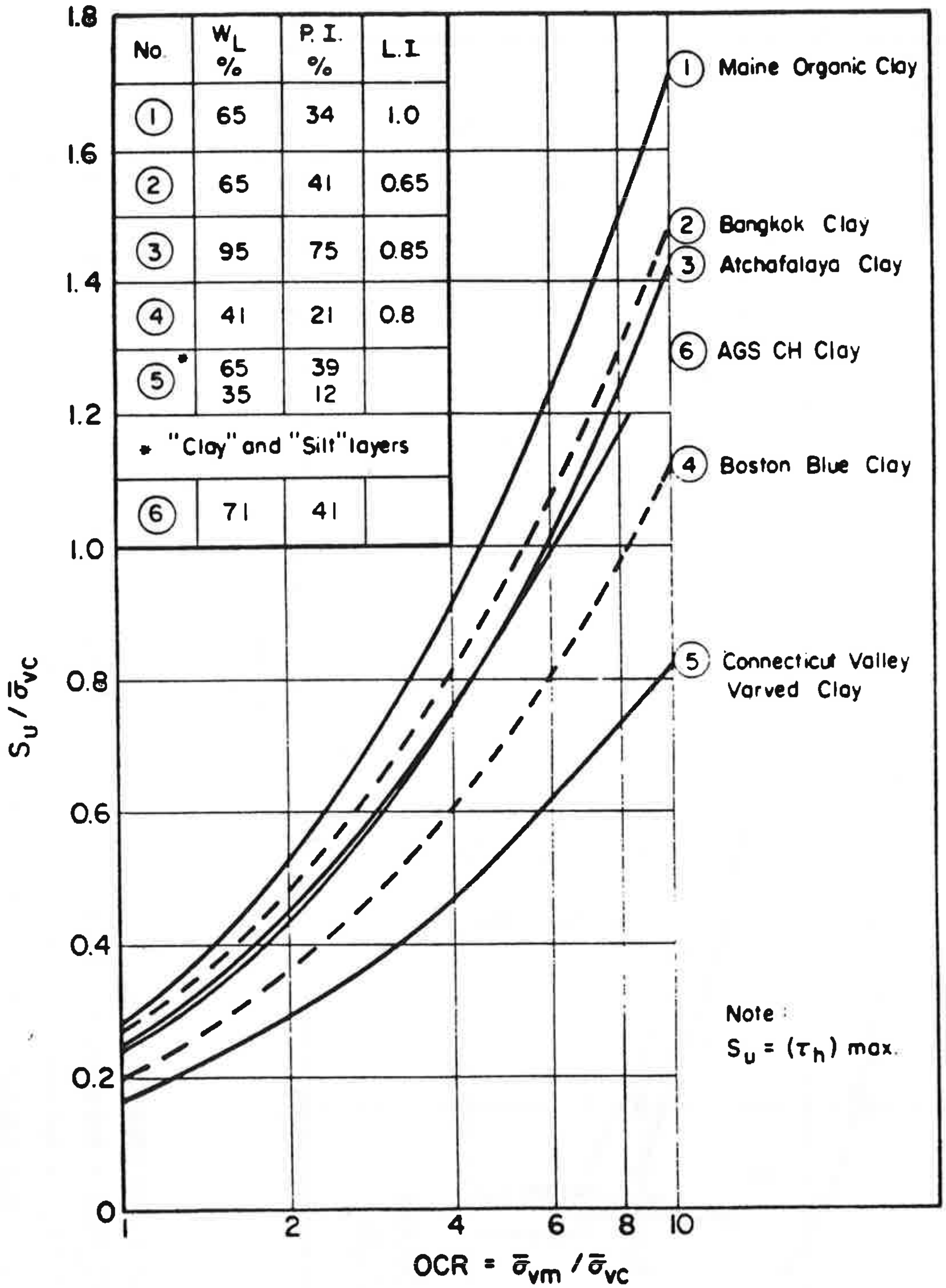
$\bar{\sigma}_{vc}$ = In situ vertical effective stress

DSS = $\overline{CK_oU}$DSS = Anisotropically consolidated, undrained direct-simple shear test data

TC = $\overline{CK_oU}$C = Anisotropically consolidated, undrained triaxial compression test data

Reproduced from "Consolidated-Undrained Direct-Simple Shear Tests on Saturated Clays," p. 222, by C.C. Ladd and L. Edgers. Year of first publication: 1972.

FIGURE A-4.1.2a. UNDRAINED STRENGTH RATIO VERSUS PLASTICITY INDEX FROM UNDRAINED SHEAR TESTS ON NORMALLY CONSOLIDATED CLAYS

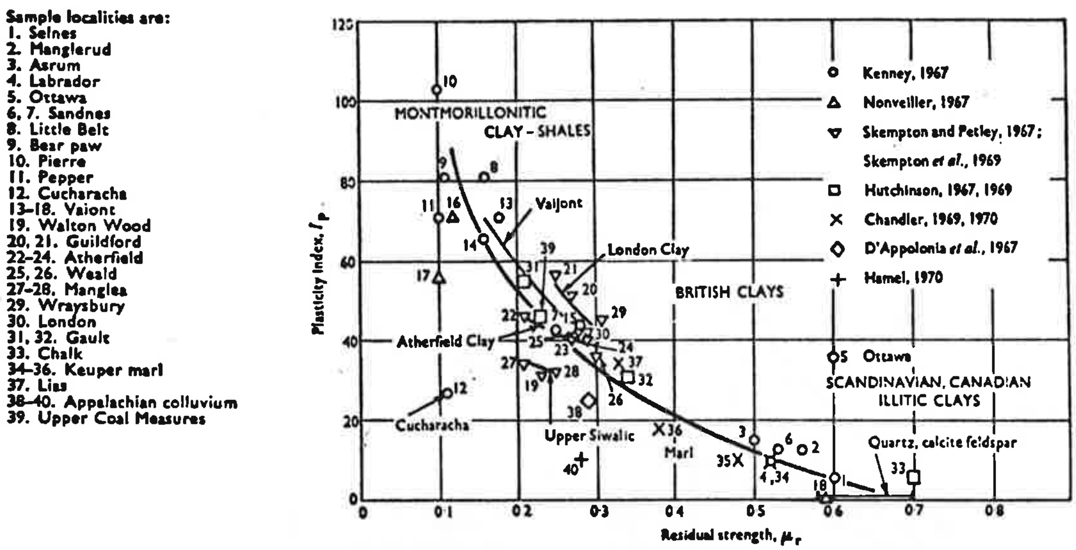


Adapted and reproduced from "Consolidated-Undrained Direct-Simple Shear Tests in Saturated Clays," p. 224, by C. C. Ladd and L. Edgers. Year of first publication: 1972.

FIGURE A-4.1.2b. UNDRAINED STRENGTH RATIO VERSUS OVERCONSOLIDATION RATIO FROM CK_0U DIRECT-SIMPLE SHEAR TESTS ON FIVE CLAYS

A4.2 Relation Between Plasticity and Residual Drained Strength

The residual drained shear strength (S_r) is the shear strength at large strain for a sample that is sheared under drained conditions. Samples can swell or shrink, changing their water content, to reach the residual state. The residual strength factor $\mu_r = S_r / \bar{\sigma}_{vc}$, where $\bar{\sigma}_{vc}$ represents the effective confining stress, has been related to the plasticity index as shown in Figure A-4.2. μ_r represents the tangent of the residual effective stress friction angle ($\bar{\Phi}_r$); i.e., $\tan \bar{\Phi}_r = \mu_r$



Reproduced from "Correlation Between Atterberg Plasticity Limits and Residual Strength of Natural Soils," p. 266, by B. Voight by permission of the Institution of Civil Engineers. Year of publication: 1973.

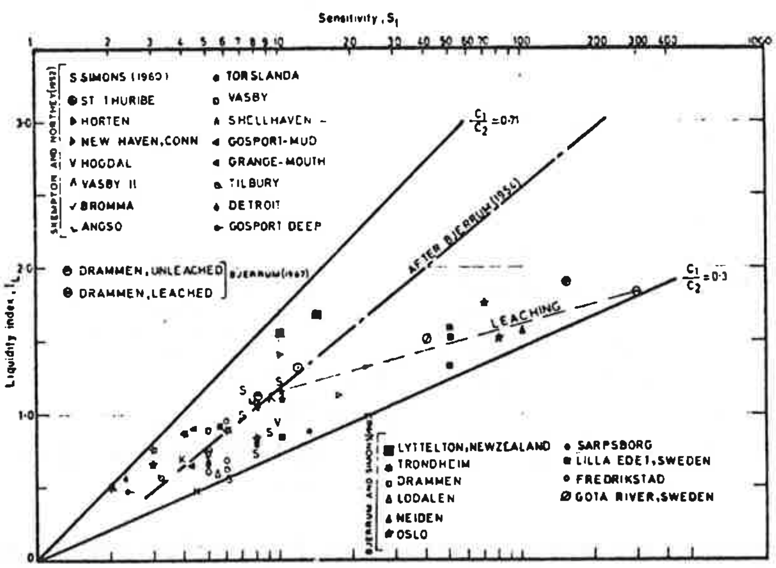
FIGURE A-4.2. RESIDUAL STRENGTH COEFFICIENT VERSUS PLASTICITY INDEX

A4.3 Relation Between Sensitivity and Liquidity Index

Sensitivity expresses the ratio between the undisturbed and the remolded undrained shear strength for soils. The liquidity index, I_L, represents the relation between the natural water content and the plasticity indices.

$$I_L = (W_n - W_P) / (W_L - W_P)$$

where W_n represents the natural water content and W_P and W_L represent the plastic and liquid limits, respectively. The relation between sensitivity and liquidity index is shown in Figure A-4.3.

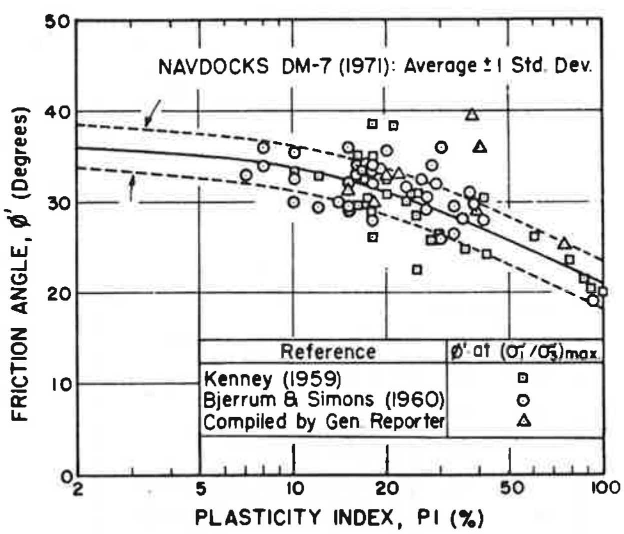


Reproduced from "Field Compressibility of Soft Sensitive Normally Consolidated Clays," p. 36, by Yudbir by permission of the Southeast Asian Society of Soil Engineering. Year of first publication: 1973.

FIGURE A-4.3. UNDRAINED STRENGTH SENSITIVITY VERSUS LIQUIDITY INDEX

A4.4 Relation Between Effective Friction Angle and Plasticity

The effective stress friction angle of cohesive soils ($\overline{\Phi}$) has been observed to be a general function of plasticity, according to Soil Mechanics, Foundations, and Earth Structure, U.S. NAVFAC Design Manual DM-7. This relation and some data from triaxial compression tests showing the general trend and the scatter in observed data, are shown in Figure A-4.4.



Reproduced from "Stress-Deformation and Strength Characteristics," p. 477, by C.C. Ladd et al., by permission of the Japanese Society of Soil Mechanics and Foundation Engineering. Year of first publication: 1977.

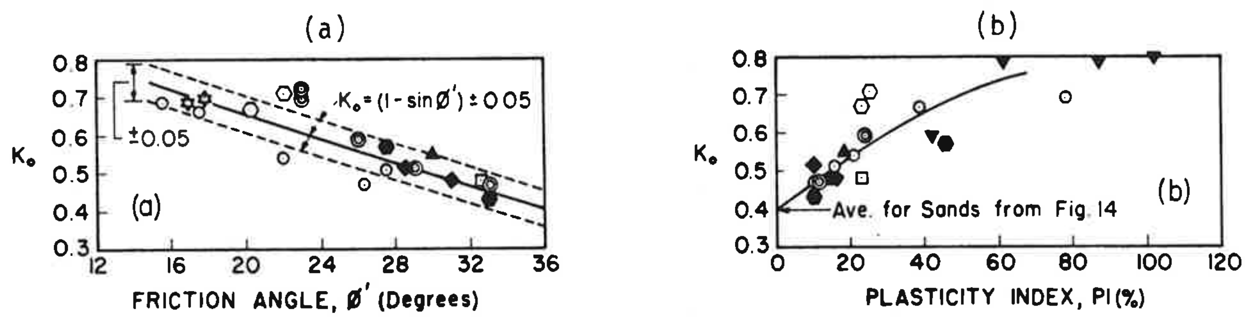
FIGURE A-4.4. EMPIRICAL CORRELATION BETWEEN FRICTION ANGLE AND PLASTICITY INDEX FOR NORMALLY CONSOLIDATED UNDISTURBED CLAYS

A4.5 Lateral Earth Pressure Coefficient

The lateral earth pressure coefficient (K_o) is the ratio of the horizontal to the vertical effective stress; i.e., $K_o = \bar{\sigma}_h / \bar{\sigma}_v$, where $\bar{\sigma}_h$ and $\bar{\sigma}_v$ represent the horizontal and vertical effective stresses, respectively.

A4.5.1 Normally Consolidated Clays

The lateral earth pressure for normally consolidated clays, as measured in the laboratory on both remolded and undisturbed clays, can be correlated with both friction angle ($\bar{\Phi}$) and plasticity index (PI), as shown in Figure A-4.5.1.



Key

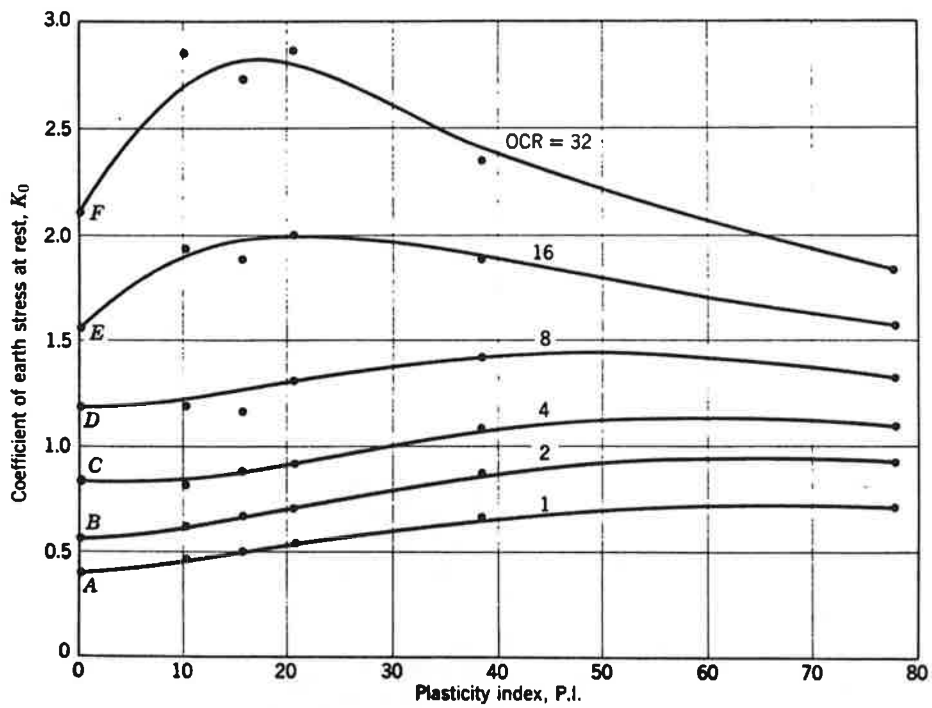
Re-molded	Un-disturbed	REFERENCE
○		Brooker & Ireland (1965)
□	■	R Ladd (1965)
⬡	⬢	Bishop (1958)
	◆	Simons (1958)
	▲	Campanella & Vaid (1972)
◎		Compiled by Wroth (1972)
	▼	Geot. Eng Inc. (1976)
✡		Abdelhamid & Krizek (1976)

Reproduced from :Stress-Deformation and Strength Characteristics," p. 442, by C.C. Ladd et al., by permission of the Japanese Society of Soil Mechanics and Foundation Engineering. Year of first publication: 1977.

FIGURE A-4.5.1. LATERAL EARTH PRESSURE COEFFICIENT, K_o, VERSUS (a) FRICTION ANGLE AND (b) PLASTICITY INDEX

A4.5.2 Over consolidated Clays

The stress history of a clay is represented by the overconsolidation ratio, OCR. OCR is defined as the ratio of the maximum past effective stress ($\bar{\sigma}_{vm}$) to the existing vertical effective stress ($\bar{\sigma}_{vc}$). Thus, OCR = $\bar{\sigma}_{vm}/\bar{\sigma}_{vc}$. For normally consolidated clays, OCR = 1 by definition. The coefficient of lateral earth pressure is related to plasticity index and OCR in Figure A-4.5.2.



Reproduced from Soil Mechanics, p. 300, by T.W. Lambe and R.V. Whitman by permission for John Wiley & Sons, Inc. Year of first publication: 1969.

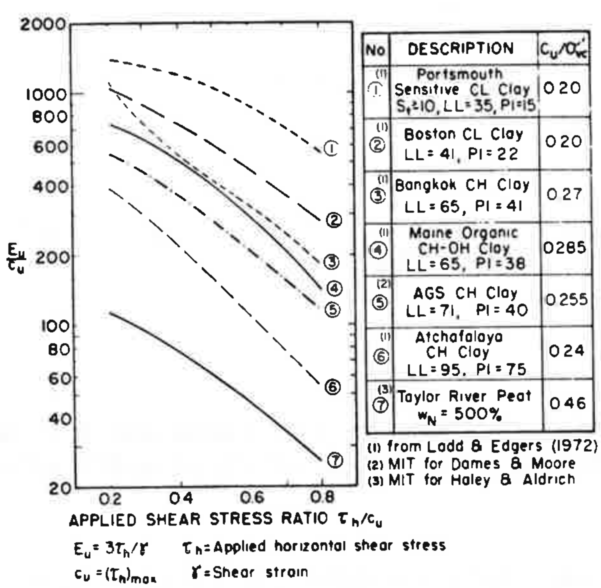
FIGURE A-4.5.2. LATERAL EARTH PRESSURE COEFFICIENT AS A FUNCTION OF OVERCONSOLIDATION RATIO AND PLASTICITY INDEX

A4.6 Laboratory and Field Studies of Equivalent Young's Modulus of Cohesive Soils

For cohesive soils, the Young's modulus may be correlated with the undrained shear strength. Since the stress-strain relation for soils is non-linear, the modulus is also a function of the applied shear stress. Figure A-4.6.1 shows the ratio of modulus to undrained shear strength for normally consolidated clays. Figure A-4.6.2 shows the relation between modulus ratio and overconsolidation ratio for two levels of applied shear stress ratio. To use these data for other clays, interpolate values based on plasticity limits.

Both figures report relations for Young's modulus. However, the modulus data were determined by measurement of undrained shear modulus from Geonor direct-simple shear tests. Shear modulus (G) and Young's modulus (E_u) are related by $E_u = 3G$ for undrained shear of clays.

Data for moduli developed from finite element analyses of actual field cases are reported by D'Appolonia et al. in Table A-4.6.3.

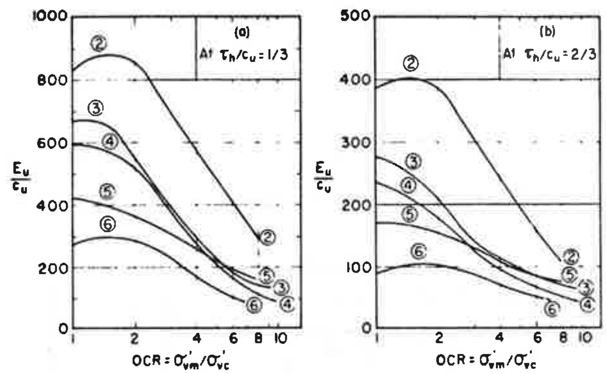


No	DESCRIPTION	c_u/σ'_{vc}
(1) 1	Portsmouth Sensitive CL Clay $S_t \doteq 10$, LL = 35, PI = 15	0.20
(1) 2	Boston CL Clay LL = 41, PI = 22	0.20
(1) 3	Bangkok CH Clay LL = 65, PI = 41	0.27
(1) 4	Maine Organic CH-OH Clay LL = 65, PI = 38	0.285
(2) 5	AGS CH Clay LL = 71, PI = 40	0.255
(1) 6	Atchafalaya CH Clay LL = 95, PI = 75	0.24
(3) 7	Taylor River Peat w_N = 500%	0.46

(1) from Ladd & Edgers (1972)
(2) MIT for Dames & Moore
(3) MIT for Haley & Aldrich

Reproduced from "Stress-Deformation and Strength Characteristics," p. 436, by C.C. Ladd et al., by permission of the Japanese Society of Soil Mechanics and Foundation Engineering. Year of first publication: 1977.

FIGURE A-4.6.1. NORMALIZED YOUNG'S MODULUS VERSUS STRESS LEVEL FOR NORMALLY CONSOLIDATED CLAYS FROM DIRECT-SIMPLE SHEAR TESTS



(See Figure A-4.6.1 for identification of soil numbers and sources of data.)

Reproduced from "Stress-Deformation and Strength Characteristics," p. 441, by C.C. Ladd et al., by permission of the Japanese Society of Soil Mechanics and Foundation Engineering. Year of first publication: 1977.

FIGURE A-4.6.2. NORMALIZED YOUNG'S MODULUS VERSUS OVERCONSOLIDATION RATIO AT TWO APPLIED STRESS LEVELS

TABLE A-4.6.3. UNDRAINED MODULUS AND MODULUS RATIO BASED ON CASE STUDIES OF INITIAL SETTLEMENT

Case No.	Location and Structure	Clay Properties			$(E_u)_{field}$ (t/m^2)	E_u/S_u	Source of S_u	Reference
			S_t	OCR				
1	Oslo - 9 story bldg.	15	2	3.5	7600	1200	CIU	Simons (1963)
2	Asrum I-Circular load Test	16	100	2.5	990	1000 1200	Field Vane CIU	Hoeg et al (1969)
3	Asrum II-Cir. load Test	14	100	1.7	880	1000 1100	Field Vane CIU	Hoeg et al (1969)
4	Mastemyr-Circular load Test	14		1.5	1300	1200 1700	Field Vane Bearing Cap.	Clausen (1969)
5	Portsmouth -Highway Embank.	15	10	1.3	3000	2000 1700	Field Vane Bearing Cap.	Haley & Aldrich, Inc. Ladd (1969)
6	Boston-Highway embank.	24	5	1.5 1.0	10,000 13,000	1600 1200 2500 1500	Field Vane CK_oU Field Vane CK_oU	MIT
7	Drammen-Cir. load Test	28	10	1.4	3200	1400 1100	Field Vane CK_oU	NGI
8	Kawasaki-Cir. load Test	38	6 ± 3	1.0	2200	400	Field Vane CIU	MIT
9	Venezuala-Oil tanks	37	8 ± 2	1.0	5000	800	CIU	Lambe (1962)
10	Maine-Rectangular load Test	33 2	4	1.5-4.5	100-200	80-160	UU & Bearing Cap.	Ladd et al (1969)

where-

PI = Plasticity index
S_t = Sensitivity
OCR = Overconsolidation ratio
$E_{u(field)}$ = Undrained secant Young's modulus for finite element analyses
S_u = Undrained shear strength
CIU = Isotropically consolidated-undrained triaxial tests
CK_oU = Anisotropically consolidated-undrained triaxial tests

From "Initial Settlement of Structures on Clay," p. 1370-1, by D.J. D'Appolonia et al. by permission of American Society of Civil Engineers. Year of first publication: 1971.

A4.7 Consolidation Compression and Swell Properties

As explained in the text, the basic equation describing the one-dimensional compression or swell of a soil is -

$$\rho = H.\frac{C}{1 + e_o} \cdot \log_{10}\left[\frac{\bar{\sigma}_{vo} + \Delta\sigma_v}{\bar{\sigma}_{vo}}\right]$$

where
ρ = settlement or swell
H = thickness of the layer of compressible soil
e_o = initial void ratio
$\bar{\sigma}_{vo}$ = initial vertical effective stress
$\Delta\sigma_v$ = change in vertical stress
C = compression or swell index

A 4.7.1 Compression of Normally Consolidated Clays

A considerable number of relations between virgin compression index (C_c) and index properties have been proposed in the past, as collected in Table A-4.7.1. Of these, the one accepted most generally is that proposed by Terzaghi and Peck for low to moderately sensitive undisturbed clays:

$$C_c = 0.009\ (w_L - 10)$$

where

w_L = liquid limit in percent.

Some relations are also shown for the compression ratio (C_r) where

$$C_r = C_c/(1+e_o)$$

The different relations apply to various types of cohesive soils.

TABLE A-4.7.1. SUMMARY OF PUBLISHED REGRESSION EQUATIONS FOR PREDICTION OF COMPRESSION INDEX, C_c, AND COMPRESSION RATIO, C_r, OF NORMALLY CONSOLIDATED SOILS

Regression Equation	Regions of Applicability	Reference
$C_c=0.007(w_L-7)$	Remolded clays	Skempton, 1944
$C_r=0.208e_o+0.0083$	Chicago clays	Peck and Reed, 1954
$C_c=17.66\times10^{-5}w_n^2+5.93\times10^{-3}w_n-0.135$	Chicago clays	Peck and Reed, 1954
$C_c=1.15(e_o-0.35)$, or $0.0054(2.6w_n-35)$	All clays	Nishida, 1956, JSMFD V82, SM3
$C_c=0.30(e_o-0.27)$	Inorganic, cohesive soil; silt, some clay; silty clay; clay	Hough, 1957
$C_c=1.15\times10^{-2}w_n$	Organic soils-meadow mats, peats, and organic silt and clay	Moran, Proctor, Mueser and Rutledge, 1958
$C_c=0.256+0.43(e_o-0.84)$	Brazilian clays	Cozzolino, 1961
$C_c=0.0046(w_L-9)$	Brazilian clays	Cozzolino, 1961
$C_c=1.21+1.055(e_o-1.87)$	Motley clays from Sao Paulo city	Cozzolino, 1961
$C_c=0.0186(w_L-30)$	Motley clays from Sao Paulo city	Cozzolino, 1961
$C_c=0.009(w_L-10)$	Undisturbed clays	Terzaghi and Peck, 1967
$C_c= 0.007(w_L-10)$	Remolded clays	Terzaghi and Peck, 1967
$C_c=0.75(e_o-0.50)$	Soils of very low plasticity	Sowers, 1970
$C_r=0.156e_o+0.0107$	All clays	Elnaggar and Krizek, 1971
$C_c=0.01w_n$	Chicago clays	Osterberg, 1972
$C_c=0.6(e_o-1)$ for $e_o < 6$	Marsh deposits, New York	Kapp et al., 1966
$C_c=0.85(e_o-2)$ for $e_o = 6$ to 14	Marsh deposits, New York	Kapp et al., 1966

where

w_n = natural water content

w_L = liquid limit

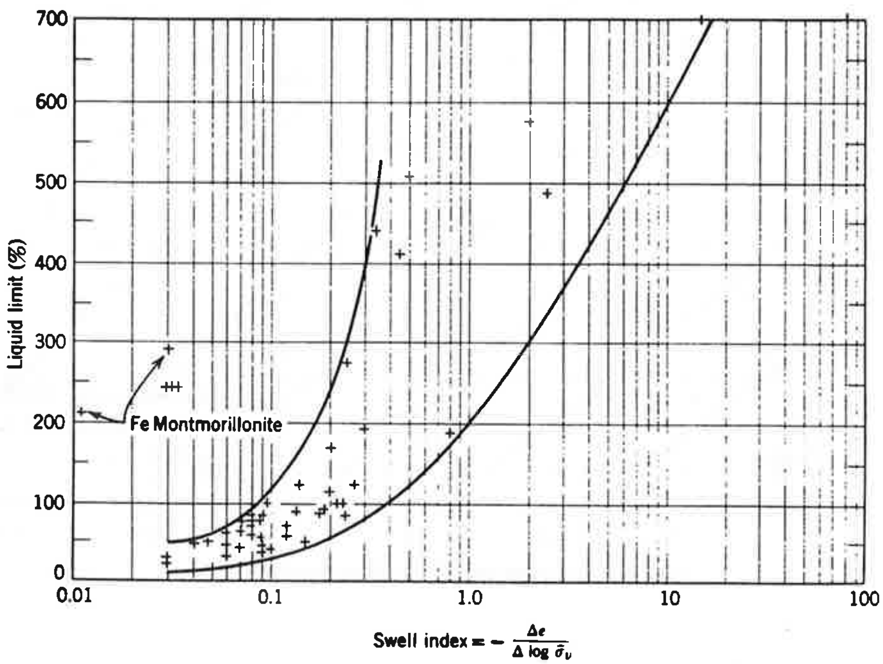
e_o = initial void ratio

From Regression Analysis of Soil Compressibility, p. 20, by A. Azzouz et al.

Year of first publication: 1976.

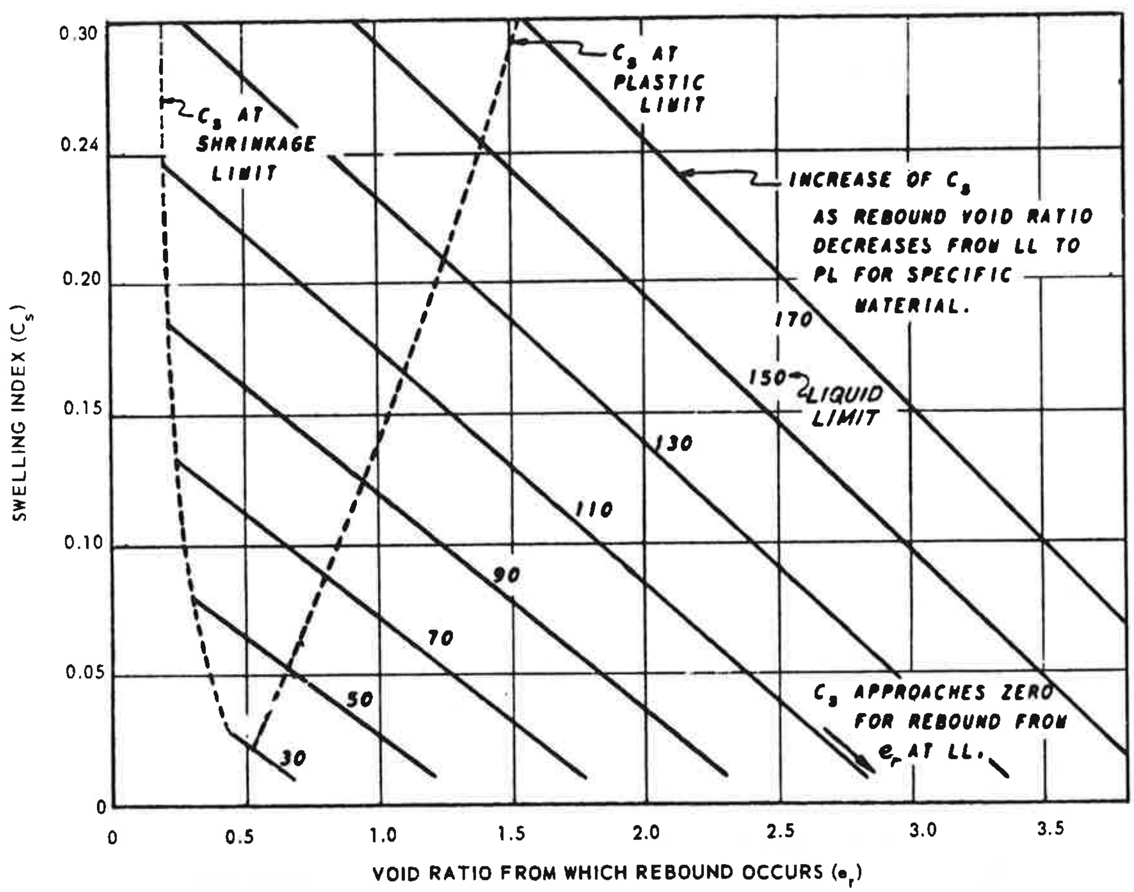
A4.7.2 Swell Properties of Clays

When a cohesive soil is unloaded, the material will swell -- producing extension strains. The same equation in 4.7.1 for settlement can be used to calculate swell; however, in this case, $C = C_s$, the swell index and ρ becomes the rise instead of the settlement. Table A-4.7.4 lists values of both compression and swell indices for various soils. Figures A-4.7.2 and A-4.7.3 relate the swell index to plasticity indices and void ratio or water content.



Reproduced from Soil Mechanics, p. 324, by T.W. Lambe and R.V. Whitman by permission of John Wiley & Sons, Inc. Year of first publication: 1969.

FIGURE A-4.7.2. SWELL INDEX VERSUS LIQUID LIMIT



Reproduced from Soil Mechanics, Foundations, and Earth Structures, p. 7-3-12, U.S.NAVFAC Design Manual DM-7. Year of first publication: 1961.

FIGURE A-4.7.3. SWELLING INDEX VERSUS REBOUND VOID RATIO AS A FUNCTION OF LIQUID LIMIT

TABLE A-4.7.4 TYPICAL VALUES FOR COMPRESSION AND SWELL INDICES OF NATURAL SOILS

Soil	Atterberg Limits W_L	W_P	Virgin Compress. Index, C_c	Swell Index, C_s 10 to 1 kg/cm^2	1 to 0.1 kg/cm^2	Reference
Expansive Soil A	84	48	-	0.14	0.25	Dawson, 1957
Expansive Soil B	87	42	0.21	0.05	0.15	Dawson, 1957
Extruded Clay Sample	47	26	0.32	0.10	0.10	Dawson, 1957
Boston Blue Clay Undisturbed	41	20	0.35	0.07	0.09	Mitchell, 1956
Boston Blue Clay Remolded	41	20	0.21	0.07	0.07	Mitchell, 1956
Fore River Clay Undisturbed	49	21	0.36	0.09	0.09	Mitchell, 1956
Fore River Clay Remolded	49	21	0.25	0.04	0.04	Mitchell, 1956
Chicago Clay Undisturbed	58	21	0.42	0.07	0.12	Mitchell, 1956
Chicago Clay Remolded	58	21	0.22	0.07	0.09	Mitchell, 1956
Louisiana Clay Undisturbed	74	26	0.33	0.05	0.08	Mitchell, 1956
Louisiana Clay Remolded	74	26	0.29	0.04	0.07	Mitchell, 1956
New Orleans Clay Undisturbed	79	26	0.29	0.04	0.08	Mitchell, 1956
New Orleans Clay Remolded	79	26	0.26	0.04	0.09	Mitchell, 1956
Montana Clay	58	28	0.21	0.04	0.07	Lambe-Martin, 1957
Fort Union Clay	89	20	0.26	0.04	--	Smith-Redlinger,1953
Beauharnois Clay	56	22	0.55	0.01	0.04	Mitchell, 1956
Cincinnati Clay	30	12	0.17	0.02	0.03	Mitchell, 1956
St. Lawrence Clay	55	22	0.84	0.04	0.08	Mitchell, 1956
Siburua Clay	70	26	0.21	0.08	0.12	Mitchell, 1956
Mississippi loess	23-43	17-29	0.09-0.23			Sheeler, 1968
Delaware organic silty clay	84	46	0.95			Schmidt & Gould,1968
Indiana silty clay	36	20	0.21			
Marine sediment, B.C. Canada	130	74	2.3			Finn et al., 1971

From Soil Mechanics, p.323, by T. W. Lambe and R. V. Whitman by permission of John Wiley & Sons, Inc. Year of first publication: 1969.

And Foundation Engineering Handbook, p. 114, by H. Winterkorn and H -Y. Fang by permission of Litton Educational Publishing, Inc. Year of first publication: 1975.

A4.8 Rate of Primary Consolidation

The rate of consolidation of soil is governed by the consolidation equations that relate percent of total consolidation to a dimensionless time factor (T). The dimensionless time factor may be related to real time by the equation -

$$t = T\,H^2/c_v$$

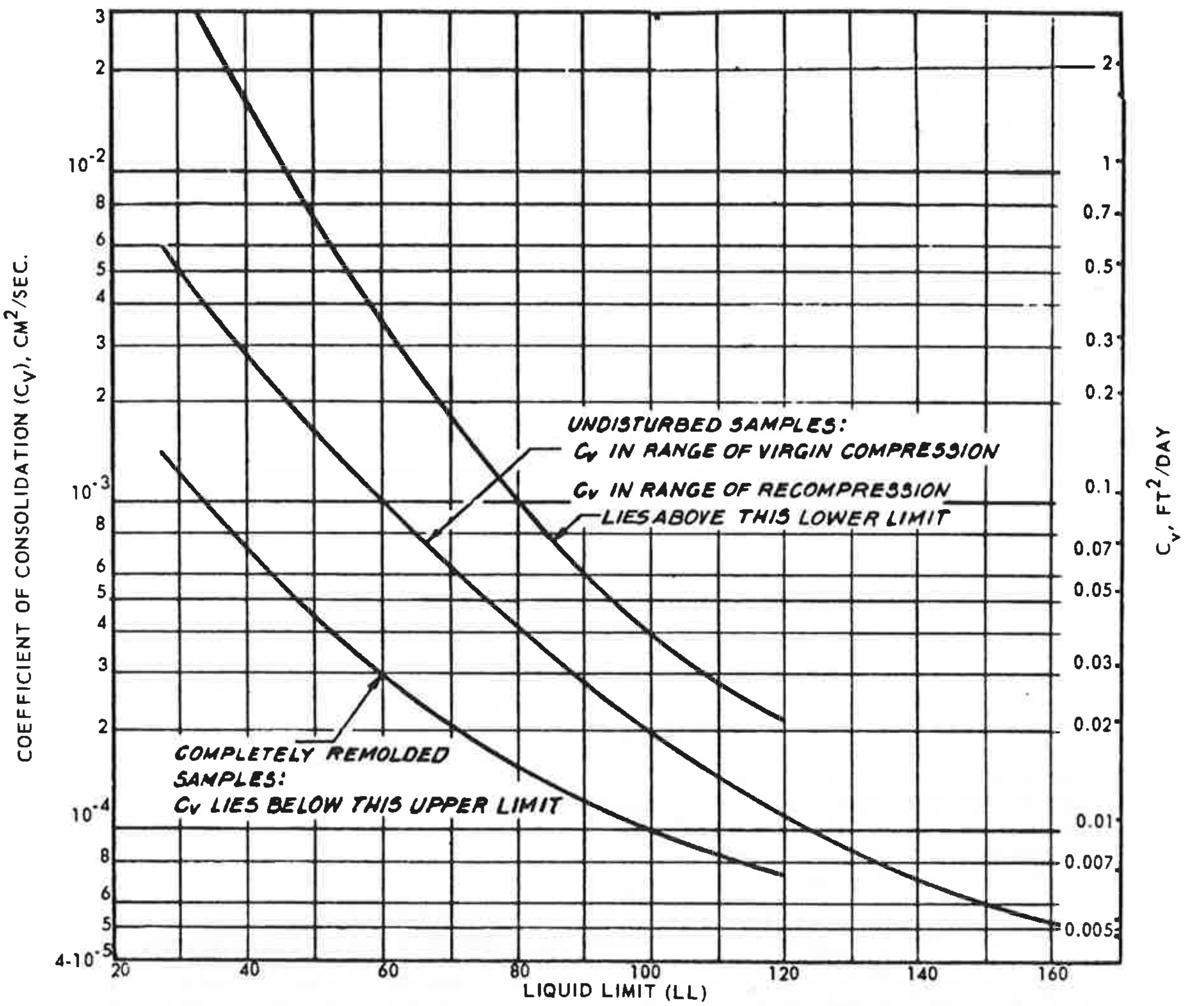
where

t = real time corresponding to the time factor, (T)

H = length of the longest distance to a drainage surface

c_v = coefficient of consolidation in units of length^2 per time.

Figure A-4.8 provides a correlation between the coefficient of consolidation and the liquid limit.



Reproduced from Soil Mechanics, Foundations, and Earth Structures, p. 7-3-14, U.S. NAVFAC Design Manual DM-7. Year of first publication: 1961.

FIGURE A-4.8. COEFFICIENT OF CONSOLIDATION, c_v, VERSUS LIQUID LIMIT WATER CONTENT FOR NORMALLY CONSOLIDATED CLAYS

A4.9 Approximate Correlations for Secondary Compression

Secondary compression refers to the continued compression of cohesive soils that takes place after all excess pore pressure has dissipated. The amount of secondary compression may be calculated by the equation:

$$\rho_s = H\, C_\alpha \log_{10}(t/t_p)$$

where-

ρ_s = Amount of predicted secondary compression

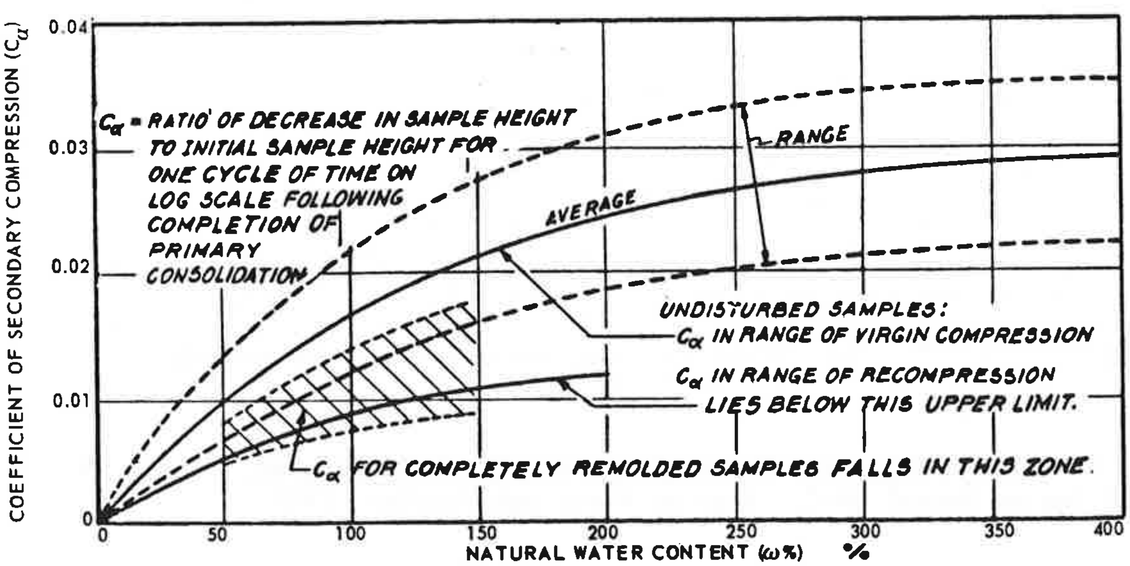
H = Thickness of clay layer

t_p = Time to complete primary consolidation

t = Time to which secondary compression is predicted

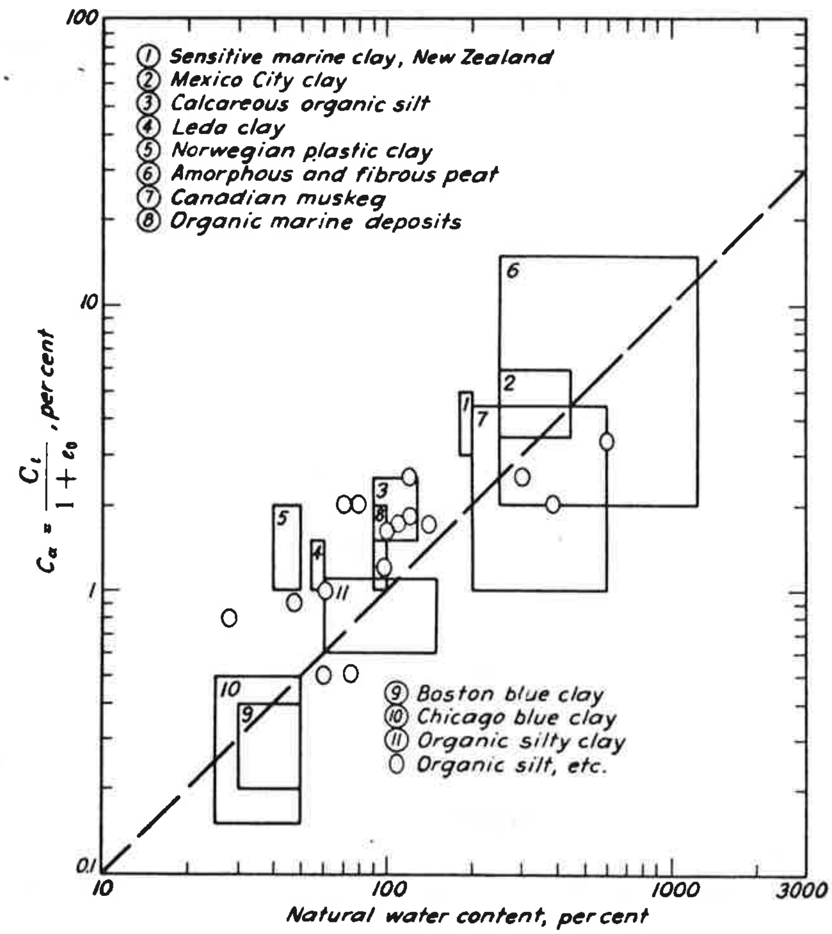
C_α = Coefficient of secondary consolidation

Figures A-4.9.1 and A-4.9.2 show approximate correlations between C_α and natural water content.



Reproduced from Soil Mechanics, Foundations, and Earth Structures, p. 7-3-14, U.S. NAVFAC Design Manual DM-7. Year of first publication: 1961.

FIGURE A-4.9.1. COEFFICIENT OF SECONDARY COMPRESSION VERSUS NATURAL WATER CONTENT



Reproduced from Foundation Engineering, Second Edition, p. 74, by R. Peck et al. by permission of John Wiley & Sons, Inc. Year of first publication: 1974.

FIGURE A-4.9.2. COEFFICIENT OF SECONDARY CONSOLIDATION VERSUS NATURAL WATER CONTENT FOR NORMALLY CONSOLIDATED CLAYS

A5. CORRELATIONS WITH FIELD TEST RESULTS

A5.1 Compressibility from Standard Penetration Tests

Several relations exist between compressibility indices and standard penetration resistance, as collected in Table A-5.1. None is generally accepted.

Reference	Relationship	Soil Types	Basis	Remarks
Schultze and Melzer (1965)	$E_s = \nu \sigma^{0.522}$ kg/cm^2 $\nu = 246.2 \log N - 263.4\ p_o + 375.6 \pm 57.6$ $0 < p_o < 1.2$ kg/cm^2 p_o = effective overburden pressure	Dry sand	Penetration tests in field and in test shaft. Compressibility based on e, $e_{max.}$ and $e_{min.}$ (Schultze and Moussa, 1961)	Correlation coefficient = 0.730 for 77 tests
Webb (1969)	$E_s = 5(N+15)$ tons/ft^2 $E_s = 10/3(N+5)$ tons/ft^2	Sand Clayey sand	Screw Plate Tests	Below water table
Farrent (1963)	$E_s = 7.5\ (1-\mu^2)N$ tons/ft^2 μ = Poisson's ratio	Sand	Terzaghi and Peck loading settlement curves	
Begemann (1974)	$E_s = 40 + C(N-6)$ kg/cm^2 $N > 15$ $E_s = C(N+6)$ kg/cm^2 $N < 15$ C = 3(silt with sand) to 12(gravel with sand)	Silt with sand to gravel with sand		Used in Greece
Trofimenkov (1974)	$E = (350\ \text{to}\ 500) \log N$ kg/cm^2	Sand		USSR practice
Meyerhof (1974)	$S = p\sqrt{B}/2N$ inches $S = p\sqrt{B}/N$ inches p in tons/ft^2, B in inches	Sand and gravel Silty sand	Analysis of field data of Schultze and Sherif (1973)	Conservative estimate of maximum settlement of shallow foundations

Note: N is penetration resistance in blows per 30 cm. (blows/ft.)

Reproduced from "In Situ Measurement of Volume Change Characteristics", p. 292, by Mitchell and Gardner by permission of A.S.C.E. Year of first publication: 1975.

FIGURE A-5.1. COMPRESSIBILITY AS INDICATED BY STANDARD PENETRATION RESISTANCE

A5.2.1 Relative Density and Penetration Resistance

The Terzaghi and Peck correlation published in Soil Mechanics in Engineering Practice, 1968, does not account for variations in overburden pressure, and is therefore approximate. This approximate but widely accepted relation is shown in Table A-5.2.1.

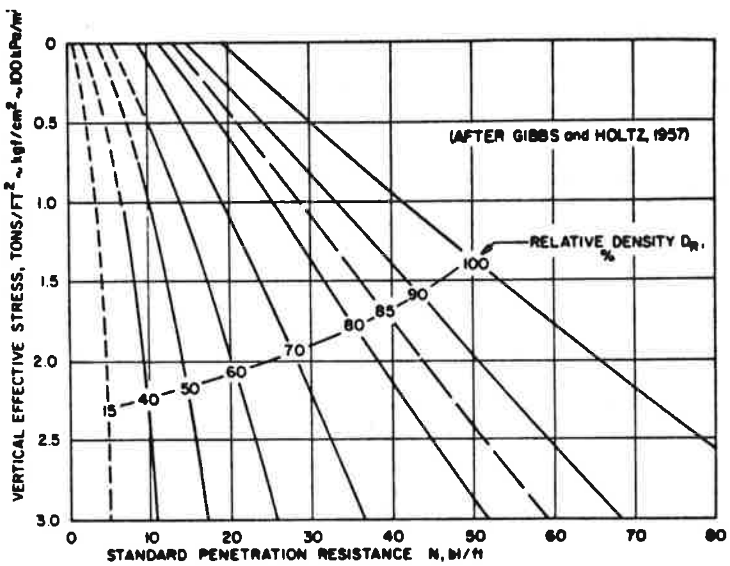
TABLE A-5.2.1. RELATION BETWEEN STANDARD PENETRATION RESISTANCE AND QUALITATIVE RELATIVE DENSITY

Penetration Resistance, N (blows/ft)	Relative Density of Sand
0-4	Very loose
4-10	Loose
10-30	Medium
30-50	Dense
Over 50	Very dense

From Soil Mechanics in Engineering Practice, second edition, p. 294, by K. Terzaghi and R. Peck, 1948. Year of first publication: 1968.

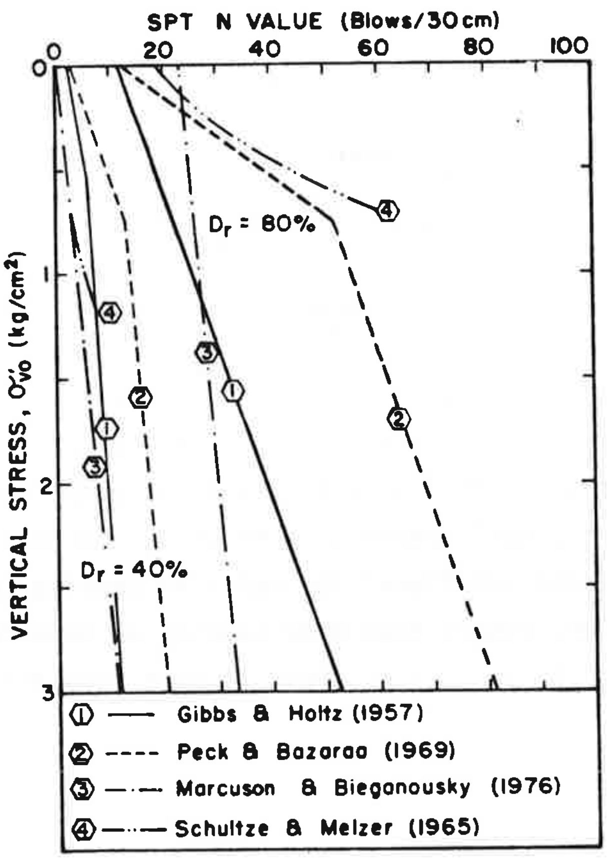
A5.2.2 Relative Density, Penetration Resistance and Overburden Pressure

Gibbs and Holtz at the Department of the Army, Waterways Experiment Station, performed a series of laboratory tests in large bins, in which they were able to measure actual density of the sand, vary the overburden pressure, and perform full size standard penetration tests. From these experiments evolved the "Gibbs & Holtz relations" for relative density, as shown in Figure A-5.2.2. However, others have made studies of penetration resistance and relative density. The results of these studies vary and are compared in Figure A-5.2.3.



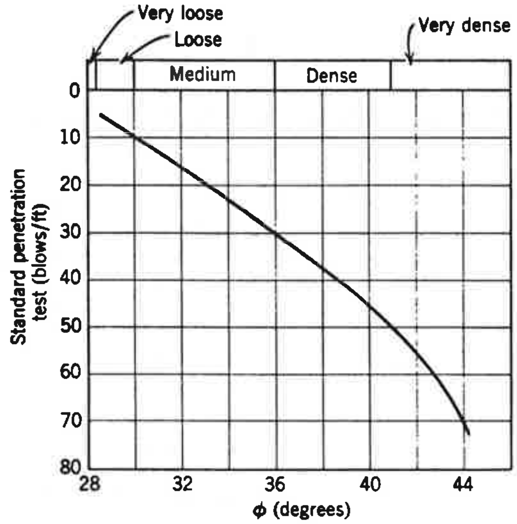
Reproduced from "In Situ Measurement of Volume Change Characteristics," p. 323, by Mitchell and Gardner by permission of A.S.C.E. Year of first publication: 1975.

FIGURE A-5.2.2. CORRELATION BETWEEN RELATIVE DENSITY AND STANDARD PENETRATION RESISTANCE ACCORDING TO GIBBS AND HOLTZ ACCOUNTING FOR EFFECT OF OVERBURDEN PRESSURE



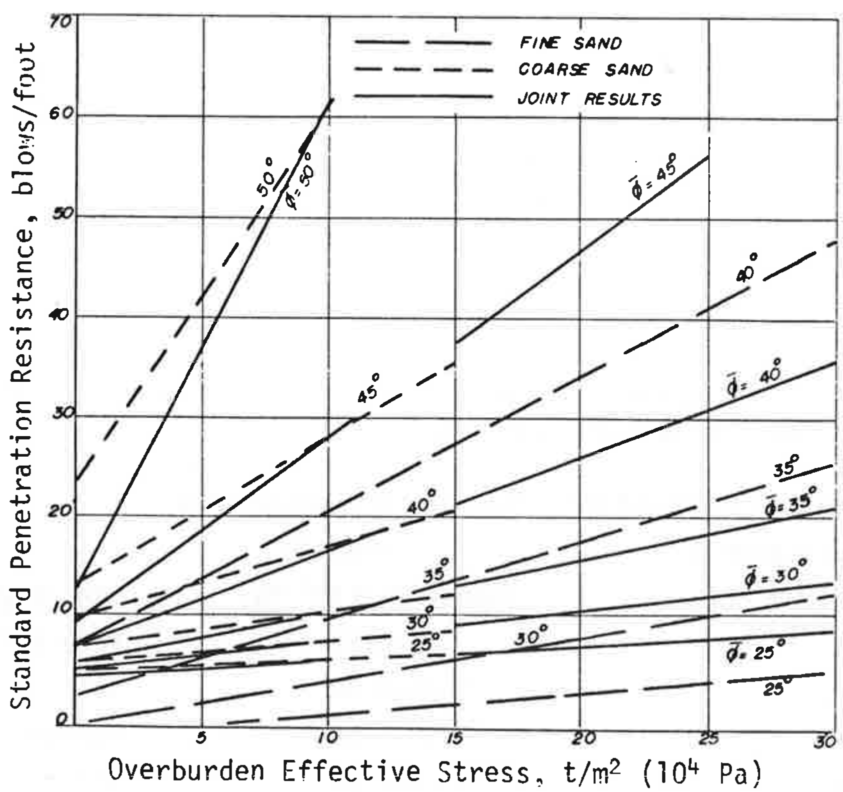
Reproduced from Stress-Deformation and Strength Characteristics," p. 465, by C. C. Ladd et al., by permission of Japanese Society of SM&FE. Year of publ.: 1977.

FIGURE A-5.2.3. COMPARISON OF CORRELATIONS BETWEEN RELATIVE DENSITY AND STANDARD PENETRATION RESISTANCE



Reproduced from Soil Mechanics, p. 148, by T.W. Lambe and R.V. Whitman by permission of John Wiley & Sons, Inc. Year of first publication: 1969.

FIGURE A-5.3.1. EARLY CORRELATION BETWEEN FRICTION ANGLE AND STANDARD PENETRATION RESISTANCE (FROM PECK, HANSON, AND THORNBURN, 1953)



Reproduced from "The Standard Penetration Test," p. 28, by V. deMello by permission of A.S.C.E. Year of first publication: 1971.

FIGURE A-5.3.2. CORRELATION AMONG FRICTION ANGLE, $\bar{\phi}$, PENETRATION RESISTANCE, AND OVERBURDEN STRESS

A5.3 Correlations for Friction Angle of Sands from Standard Penetration Resistance

In Foundation Engineering, 1953, Peck et al. provide a correlation between friction angle ($\bar{\Phi}$) and standard penetration resistance (N). This correlation, shown in Figure A-5.3.1 does not account for overburden stress and is therefore considered approximate (±3°). DeMello added the overburden stress parameter to the analysis to produce the correlation shown in Figure A-5.3.2. This latter correlation is generally preferred to the former. The deMello correlation is considerably less conservative; i.e., the correlation by Peck et al. indicates lower $\bar{\Phi}$ at the same N, except at overburden pressures greater than 2 kgf/cm^2 (200 kPa). Both correlations are judged to be conservative at overburden pressures greater than 2 kgf/cm^2 (200 kPa).

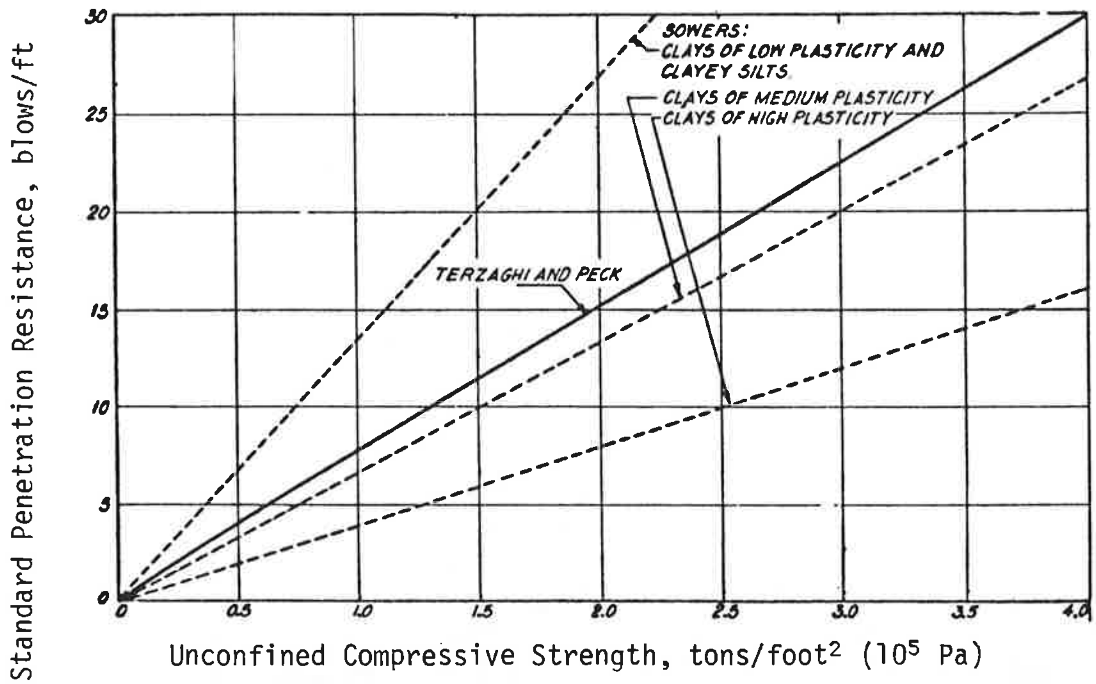
A5.4 Correlations Between Strength of Cohesive Soils and Standard Penetration Resistance

The standard penetration test is generally considered not as suitable for indicating the properties of cohesive soils as for the properties of granular soils. Correlations between unconfined compressive strength and penetration resistance have been published by Terzaghi and Peck and others as shown in Table A-5.4.1 and Figure A-5.4.2

TABLE A-5.4.1. CORRELATION BETWEEN UNCONFINED COMPRESSIVE STRENGTH AND STANDARD PENETRATION RESISTANCE

Standard Penetration Resistance N (blows/ft)	Unconfined Compressive Strength (tons/ft^2)(10^5 Pa)	Consistency
<2	<0.25	Very soft
2-4	0.25-0.50	Soft
4-8	0.50-1.00	Medium
8-15	1.00-2.00	Stiff
15-30	2.00-4.00	Very stiff
>30	>4.00	Hard

From Soil Mechanics in Engineering Practice, p. 300, by Terzaghi & Peck. Year of first publication: 1948.

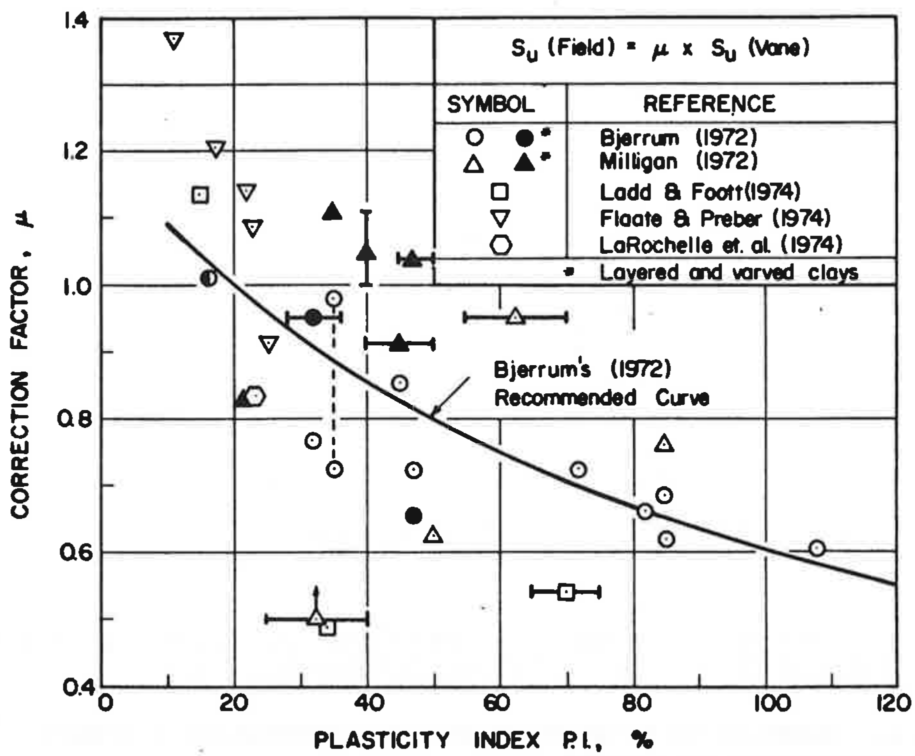


Reproduced from Soil Mechanics, Foundations, and Earth Structures, p. 7-4-7, U.S. NAVFAC Design Manual DM-7. Year of first publication: 1961.

FIGURE A-5.4.2. CORRELATION BETWEEN UNCONFINED COMPRESSIVE STRENGTH AND STANDARD PENETRATION RESISTANCE

A5.5 Field Vane Correction Factor versus Plasticity Index

The field vane test should be considered an index of the undrained shear strength rather than a direct measurement. Based on a number of studies, it has been observed that the relation between shear strength calculated from analyses of embankment failures and strength measured by the field vane is a function of the plasticity index, as shown in Figure A-5.5. These correlations are appropriate for vertical loading-type problems, but may not be applicable for other stress systems, such as in the case of excavation.

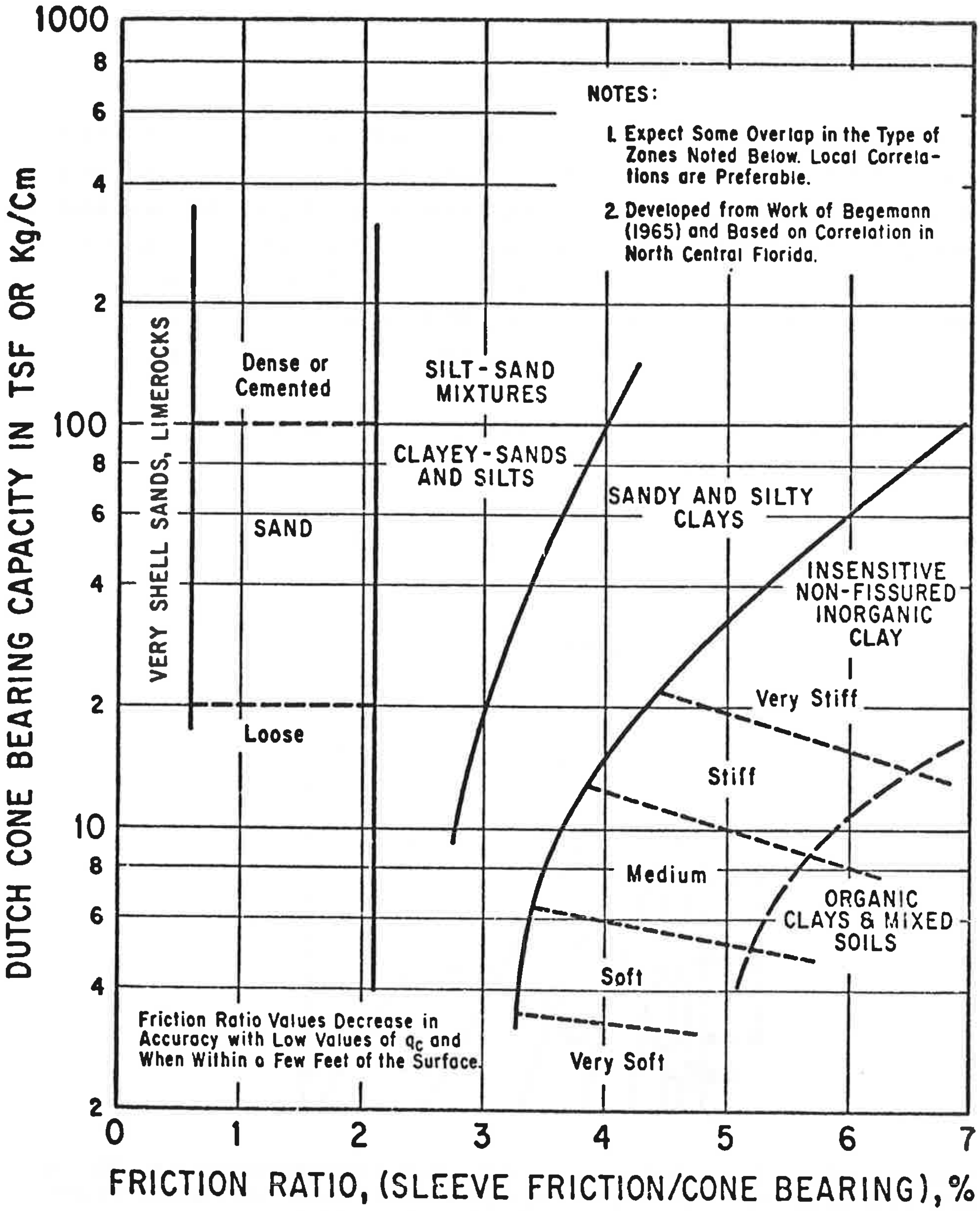


where –

S_u = Undrained Shear Strength

Reproduced from Discussion by C.C. Ladd of "Measurement of In Situ Shear Strength," p. 158, by J. Schmertmann by permission of A.S.C.E. Year of first publication: 1975.

FIGURE A-5.5. FIELD VANE CORRECTION FACTOR VERSUS PLASTICITY INDEX DERIVED FROM EMBANKMENT FAILURES



Reproduced from Guidelines for Cone Penetration Test, p. 7, by U.S.D.O.T., Federal Highway Administration. Year of first publication: 1977.

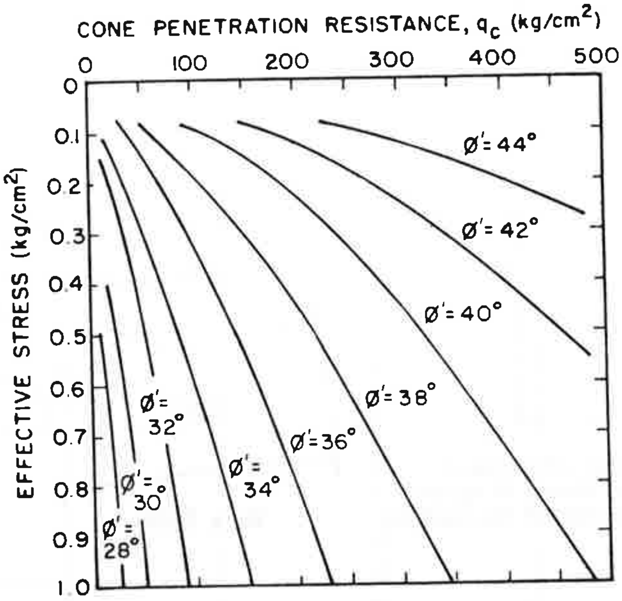
FIGURE A-5.6. GUIDE FOR ESTIMATING SOIL TYPE FROM DUTCH FRICTION-CONE RATIO FOR BEGEMANN MECHANICAL TIP

A5.6 Static Cone Penetration Test

The static or Dutch cone penetration test with friction sleeve has been suggested for use in the exploration of both cohesive and cohesionless soils by the FHWA and others. Although correlations are available, they have been developed based on limited data from locations in North America. Therefore, the use of these correlations should be confirmed by local experience or at least compared with other measures or engineering judgment. Figure A-5.6 provides a guide for estimating soil type from cone penetration data.

A5.7 Correlations Between Friction Angle and Cone Resistance

A correlation among friction angle, vertical effective stress, and cone penetration resistance as developed in the Soviet Union is presented in Figure A-5.7.



Reproduced from "Stress-Deformation and Strength Characteristics," p. 467, by C.C. Ladd et al. by permission of the Japanese Society of Soil Mechanics and Foundation Engineering. Year of first publication: 1977.

FIGURE A-5.7. CORRELATION AMONG FRICTION ANGLE, VERTICAL EFFECTIVE STRESS, AND CONE PENETRATION RESISTANCE

A5.8 Correlation Between Relative Density of Sands and Cone Resistance

A correlation among the relative density of sand, vertical effective stress, and static cone bearing resistance is shown in Figure A-5.8.

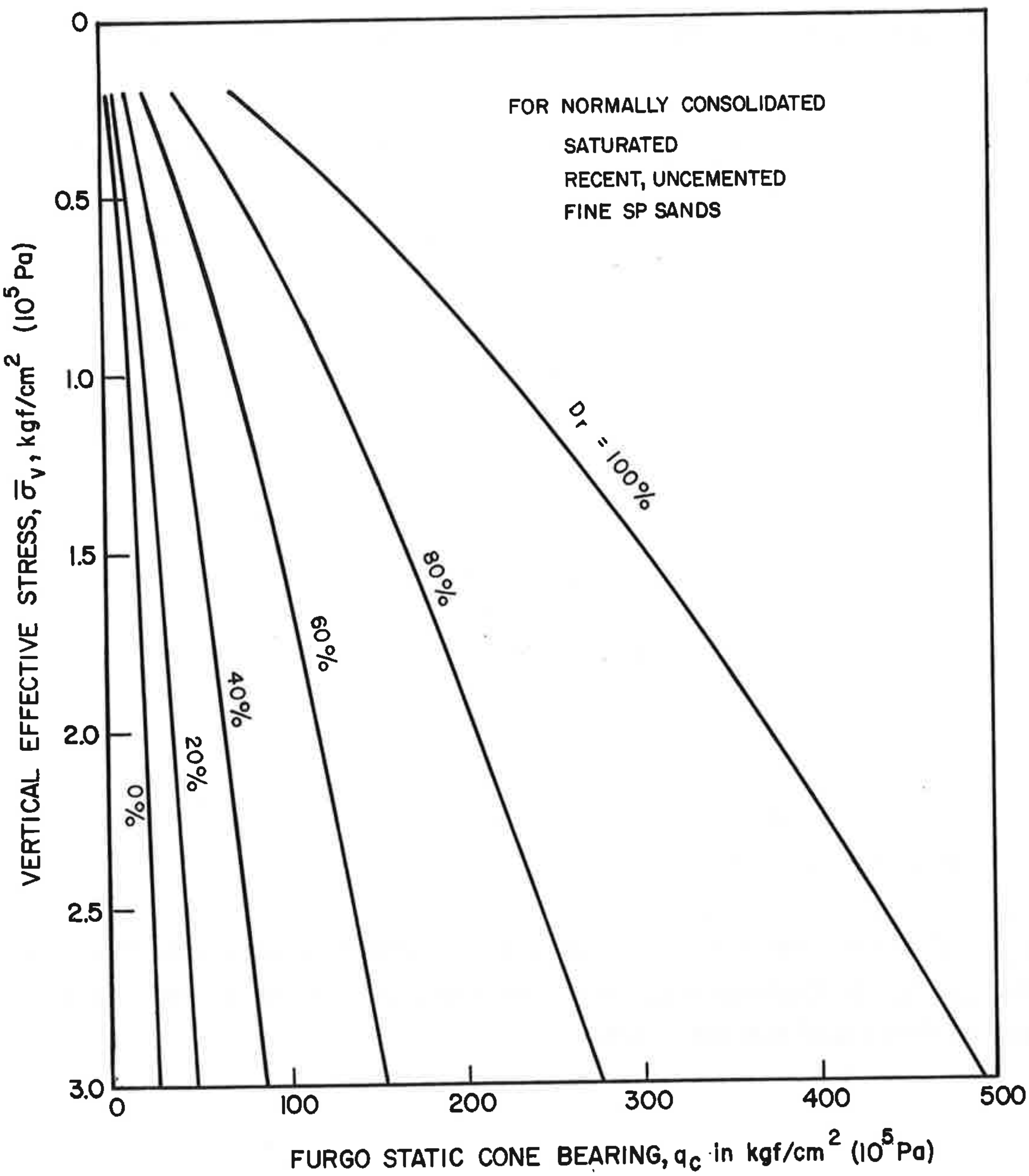
A5.9 Correlations Between Cone Penetration and Standard Penetration Resistance

Because of the similarity in application between the two common penetration tests, several correlations have been suggested to compare CPT resistance and SPT resistance, as shown in Table A-5.9.1 and Figure A-5.9.2.

TABLE A-5.9.1. APPROXIMATE RELATION BETWEEN DUTCH-CONE AND STANDARD PENETRATION RESISTANCE

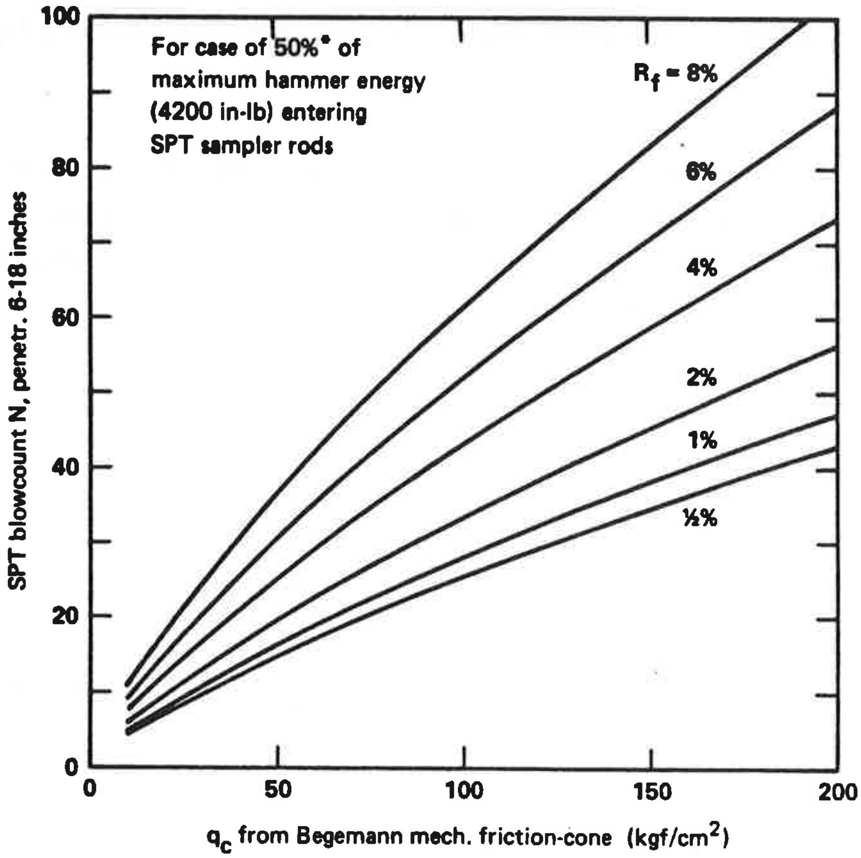
Soil Type	q_p/N*
Silts, sandy silts, slightly cohesive silt-sand mixtures	2.0
Clean fine to medium sands and slightly silty sands	3 to 4
Coarse sands and sands with little gravel	5 to 6
Sandy gravels and gravels	8 to 10

*q_p = Dutch cone resistance, kg/cm^2; N = standard penetration blows per foot
From Foundation Engineering, second edition, p. 115, by R. Peck et al.
Year of first publication: 1974.



From Guidelines for Cone Penetration Test, p. 14, U.S.D.O.T. Federal Highway Administration. Year of publication: 1977.

FIGURE A-5.8. RELATIVE DENSITY OF SANDS FROM CONE PENETRATION RESISTANCE



R_f = ratio of sleeve friction to point bearing resistance

(* Current data indicates 50% approximate U.S. average.)

Reproduced from Guidelines for Cone Penetration Test, p. 21, by U.S.D.O.T., Federal Highway Administration. Year of first publication: 1977.

FIGURE A-5.9.2. EXPERIMENTAL-THEORETICAL RELATIONSHIP BETWEEN q_c AND N USING LINER SPT SAMPLER WITHOUT LINERS AND DELFT MECHANICAL CONE

A5.10 Correlations Between Undrained Shear Strength and Cone Penetration Resistance

Several researchers have proposed correlations between undrained shear strength (S_u) and point resistance (q_c), derived from the cone penetration test. In 1969, deMello reviewed several studies in the paper, "Foundations of Buildings in Clay." These studies indicate that:

$$S_u = q_p/N_{cp}$$

where

q_p = cone point resistance

N_{cp} = empirical factor ranging from 8 to 30

In "Guidelines for the Cone Penetration Test", 1977, Schmertman suggests the relation:

$$S_u = \frac{q_p - \sigma_v}{N_c}$$

where

σ_v = total vertical stress at the test location

N_c = empirical factor

The factor N_c has been shown theoretically to be a function of the sensitivity of the clay, ranging from about 10 for an insensitive clay to 1 for a highly sensitive clay. The factor has also been shown to vary with the rate of penetration, the type of cone, and the plasticity of the clay. Values of N_c = 10 for electrical tips and N_c = 16 for Dutch mechanical tips have been suggested. However, values of 6 to 20 have been determined based on comparisons between cone penetration and field vane tests. Therefore, it is necessary to determine local correlations in order to use cone penetration data to determine shear strength with reasonable confidence.

A5.11 Compressibility from Cone Penetration Tests

There are many correlations between compressibility parameters and cone penetration resistance, as collected in Table A-5.11. None is generally accepted; few have been developed based on experience in North America.

TABLE A-5.11. COMPRESSIBILITY AS INDICATED BY STATIC CONE RESISTANCE

Reference	Relationship	Soil Types	Remarks
Buisman (1940)	$E_s = 1.5\ q_c$	Sands	Overpredicts settlements by a factor of about two
Trofimenkov (1964)	$E_s = 2.5\ q_c$	Sand	Lower limit
	$E_s = 100 + 5\ q_c$		Average
De Beer (1967)	$E_s = 1.5\ q_c$	Sand	Overpredicts settlements by a factor of two
Schultze and Melzer (1965)	$E_s = \frac{1}{m_v}\ \nu\sigma^{0.522}$	Dry sand	Based on field and lab penetration tests compressibility based on e, e_{max} and e_{min}
	$\nu = 301.1 \log q_c - 382.3\ p_o + 60.3 \pm 50.3$		Correlation coefficient = 0.778 for 90 tests valid for p_o = 0 to 0.8 kg/cm²
Bachelier and Parez (1965)	$E_s = \alpha q_c$		
	$\alpha = 0.8\text{-}0.9$	Pure sand	
	$\alpha = 1.3\text{-}1.9$	Silty sand	
	$\alpha = 3.8\text{-}5.7$	Clayey sand	
	$\alpha = 7.7$	Soft clay	
De Beer (1967)	$A = C\ \frac{A_{oed}}{C_{oed}}$	Overconsolidated sand	C from field tests A_{oed} and C_{oed} from lab oedometer tests $C_{oed} = 2.3\ \frac{(1+e)}{C_c}$ $A_{oed} = 2.3\ \frac{(1+e)}{C_s}$
Thomas (1968)	$E_s = \alpha q_c$ $\alpha = 3 - 12$	3 sands	Based on penetration and compression tests in large chambers Lower values of α at higher values of q_c; attributed to grain crushing
Webb (1969)	$E_s = \frac{5}{2}(q_c + 30)$ tsf	Sand below water table	Based on screw plate tests Correlated well with settlement of oil tanks
	$E_s = \frac{5}{3}(q_c + 15)$ psf	Clayey sand below water table	
Meigh and Corbett (1969)	$E_s = \frac{1}{m_v} = \alpha q_c$	Soft silty clay	See Fig. 2 and text
Vesić (1970)	$E_s = 2(1 + D_R^{\ 2})q_c$ D_R = relative density	Sand	Based on pile load tests and assumptions concerning state of stress
Schmertmann (1970)	$E_s = 2q_c$	Sand	Based on screw plate tests Δσ = 2 tsf

TABLE A-5.11. (Continued)

Reference	Relationship	Soil Types	Remarks
Gielly et al. (1969), Sanglerat et al. (1972)	$E_s = \alpha q_c$		Based on 600 comparisons between field penetration and lab oedometer tests
	$q_c < 7$ bars, $3 < \alpha < 8$	Clays of low plasticity (CL)	
	$7 < q_c < 20$ bars, $2 < \alpha < 5$		
	$q_c > 20$ bars, $1 < \alpha < 2.5$		
	$q_c > 20$ bars, $3 < \alpha < 6$	Silts of low plasticity (ML)	
	$q_c < 20$ bars, $1 < \alpha < 3$		
	$q_c < 20$ bars, $2 < \alpha < 6$	Highly plastic silts and clays (MH,CH)	
	$q_c < 12$ bars, $2 < \alpha < 8$	Organic silts (OL)	
	$q_c < 7$ bars:		
	$50 < w < 100$, $1.5 < \alpha < 4$	Peat and organic clay (Pt, OH)	
	$100 < w < 200$, $1 < \alpha < 1.5$		
	$w > 200$, $0.4 < \alpha < 1$		
	$20 < q_c < 30$ bars, $2 < \alpha < 4$	Gravel	
	$q_c > 30$ bars, $1.5 < \alpha < 3$		
	$q_c < 50$ bars, $\alpha = 2$	Sand	
	$q_c > 100$ bars, $\alpha = 1.5$		
	$q_c > 12$ bars, $w < 30\%$, $C_c < 0.2$		See Fig. 9
	$q_c < 12$ bars, $w < 25\%$, $C_c < 0.2$		
	$25 < w < 40\%$, $0.2 < C_c < 0.3$		
	$40 < w < 100\%$, $0.3 < C_c < 0.7$		
	$q_c < 7$ bars, $100 < w < 130\%$, $0.7 < C_c < 1$		
	$w > 130$, $C_c > 1$		
Bogdanović (1973)	$E_s = \alpha q_c$		Based on analysis of silo settlements over a period of 10 years
	$q_c > 40$ kg/cm², $\alpha = 1.5$	Sands, sandy gravels	
	$20 < q_c < 40$, $\alpha = 1.5 - 1.8$	Silty saturated sands	
	$10 < q_c < 20$, $\alpha = 1.8 - 2.5$	Clayey silts with silty sand and silty saturated sands with silt	
	$5 < q_c < 10$, $\alpha = 2.5 - 3.0$		
Schmertmann (1974a)	$E_s = 2.5\, q_c$	NC sands	L/B = 1 to 2 axisymmetric
	$E_s = 3.5\, q_c$	NC sands	L/B ≥ 10 plane strain
De Beer (1974b)	$C > \frac{3}{2} \frac{q_c}{\sigma_o}$	NC sands	Belgian practice
	$A > \epsilon \frac{3}{2} \frac{q_c}{\sigma_o}$	OC sands	$3 < \epsilon < 10$, Belgian practice
	$E_s = 1.6\, q_c - 8$	Sand	Bulgarian practice
	$E_s = 1.5\, q_c$, $q_c > 30$ kg/cm²; $E_s = 3\, q_c$, $q_c < 30$ kg/cm²	Sand	Greek practice
	$E_s > 3/2\, q_c$ or $E_s = 2\, q_c$	Sand	Italian practice
	$E_s = 1.9\, q_c$	Sand	South African practice
	$E_s = \frac{5}{2}(q_c + 3200)$ kN/m²	Fine to medium sand	
	$E_s = \frac{5}{3}(q_c + 1600)$ kN/m²	Clayey sands, PI < 15%	
	$E_s = \alpha q_c$, $1.5 < \alpha < 2$	Sands	U.K. practice
Trofimenkov (1974)	$E_s = 3\, q_c$	Sands	U.S.S.R. practice
	$E_s = 7\, q_c$	Clays	
Meyerhof (1974)	$S = pB/2\, q_c$ in consistent units; S = settlement	Cohesionless soil	Conservative estimate, based on analysis of vertical strain
Alperstein and Leifer (1975)	$E = (11 - 22)\, q_c$	Overconsolidated sand	E_s determined by lab tests on reconstituted samples of sand
Dahlberg (1974)	$E = \alpha q_c$, $1 < \alpha < 4$	NC and OC sand	E_s back-calculated from screw plate settlement using Buisman-DeBeer and Schmertmann methods; α increases with increasing q_c: see text

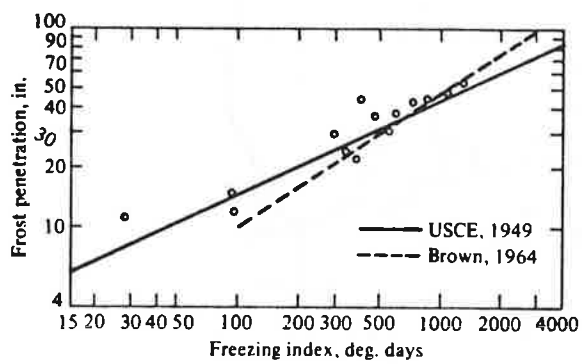
A6. FREEZE THAW RESISTANCE

A6.1 Depth of Freezing

The principal published research on the effects of frost on engineering structures has been sponsored by the U. S. Army Corps of Engineers, particularly the Cold Regions Research Engineering Laboratory (CRREL) and the British Transport Road Research Laboratory (TRRL). The depth of frost penetration below the surface is determined by the duration of subfreezing temperatures and the amount of water contained in the subsoil. The average depth of frost penetration beneath the surface of pavements kept clear of snow is shown in Figure A-6.1.

A6.2 Relation Between Depth of Frost and Freezing Index

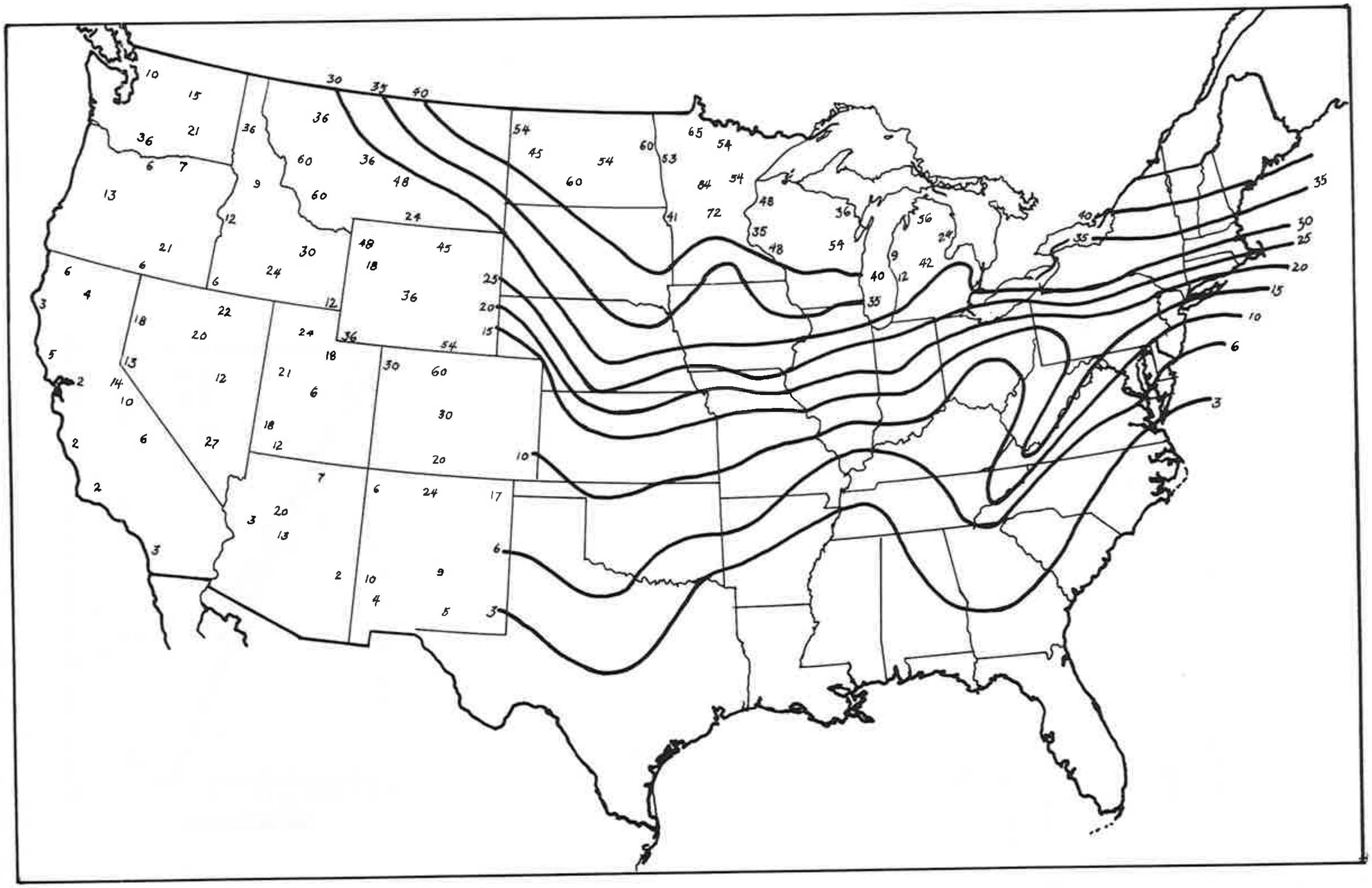
The depth of freezing has been correlated with climatic data expressed as a freezing index in degree-days so that the freezing index = No. of days x (average daily temperature - 32^{o}F.) The freezing index is determined as the number of degree-days between the highest and lowest point on a curve of cumulative degree days versus time for one complete freezing season. The design freezing index is normally for the one coldest year in 10. Local temperature variation may lead to significantly deeper or shallower frost penetration than indicated in Figure A-6.2. To convert from freezing index to depth of frost penetration, Figure A-6.1 may be used.



Freezing index = No. of days X (average daily temp. °F − 32°)

Reproduced from Foundation Engineering Handbook, p. 489, by Winterkorn and Fang by permission of Van Nostrand-Reinholt. Year of first publication: 1975.

FIGURE A-6.1. MAXIMUM FROST PENETRATION VERSUS FREEZING INDEX



(Numbers refer to frost depth in inches.)

Reproduced from Airport Paving, p. 20, by U.S. Federal Aviation Agency. Year of first publication: 1964.

FIGURE A-6.2. DEPTH OF FROST PENETRATION

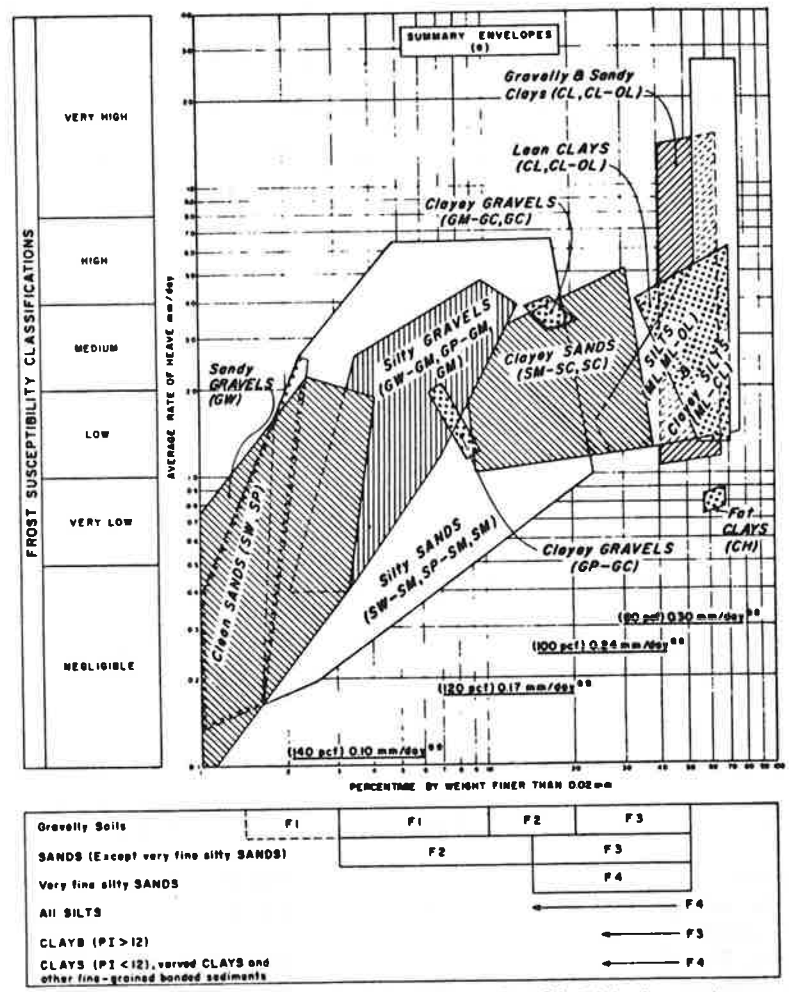
A6.3 Correlations Between Soil Classification and Frost Heave Potential

Based on laboratory and field tests of the frost heave of soil subgrades below pavements, the U. S. Army Arctic Construction and Frost Effects Laboratory (now CRREL) developed correlations between Unified soil type, percent of material finer than 0.02mm, and susceptibility to frost heave. There are four frost groups with higher-numbered groups indicating higher frost heave rates and potential for damage. These frost groups are often used in conjunction with pavement design procedures. The correlation between Unified soil type and the frost group are presented in Table A-6.3.1. The correlation between the rate of heave and percentage finer than 0.02mm for the various frost types is shown in Figure A-6.3.2.

TABLE A-6.3.1. FROST DESIGN SOIL CLASSIFICATION

Frost Group	Soil Type	Percentage Finer Than 0.02 mm by Weight	Typical Soil Types Under Unified Soil Classification System
F1	Gravelly	3 to 10	GW, GP, GW-GM, GP-GM
F2	(a) Gravelly	10 to 20	GM, GW-GM, GP-GM
	(b) Sands	3 to 15	SW, SP, SM, SW-SM SP-SM
F3	(a) Gravelly	>20	GM, GC
	(b) Sands, except very fine silty sands	>15	SM, SC
	(c) Clays, PI >12	—	CL, CH
F4	(a) All silts	—	ML, MH
	(b) Very fine silty sands	>15	SM
	(c) Clays, PI<12	—	CL, CL-ML
	(d) Varved clays and other fine-grained, banded sediments	—	CL and ML; CL and ML and SM; CL, CH and ML; CL, CH, ML and SM

Reproduced from "Corps of Engineers' Pavement Design ...," p. 95, by K.A. Linell et al. Year of first publication: 1963.



NOTES: Standard tests performed by Arctic Construction and Frost Effects Laboratory, specimens 6 in. dia. by 6 in. high, frozen at penetration rate of approximately 0.25 in. per day, with free water at 38°F continuously available at base of specimen. Specimens compacted to 95% or better of applicable standard, except undisturbed clays. Saturations before freezing generally 85% or greater.

* Undisturbed specimen

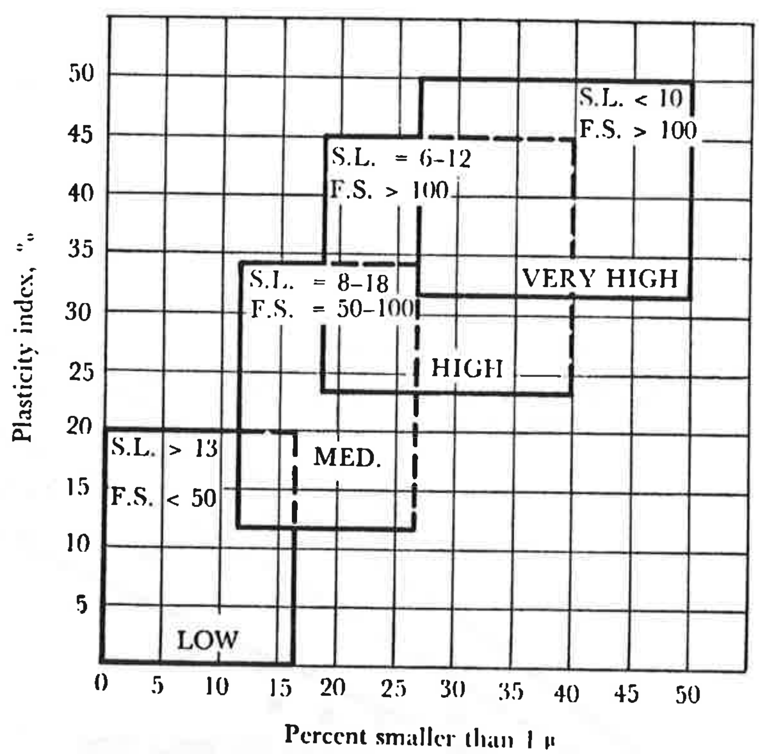
** Indicated heave rate due to expansion in volume, if all original water in 100% saturated specimen were frozen, with rate of frost penetration 0.25 inch per day.

Reproduced from "Corps of Engineers' Pavement Design ...," p. 94, by K.A. Linell et al. Year of first publication: 1963.

FIGURE A-6.3.2. AVERAGE RATE OF FROST HEAVE VERSUS PERCENT FINER THAN 0.02mm

A7. SWELL POTENTIAL

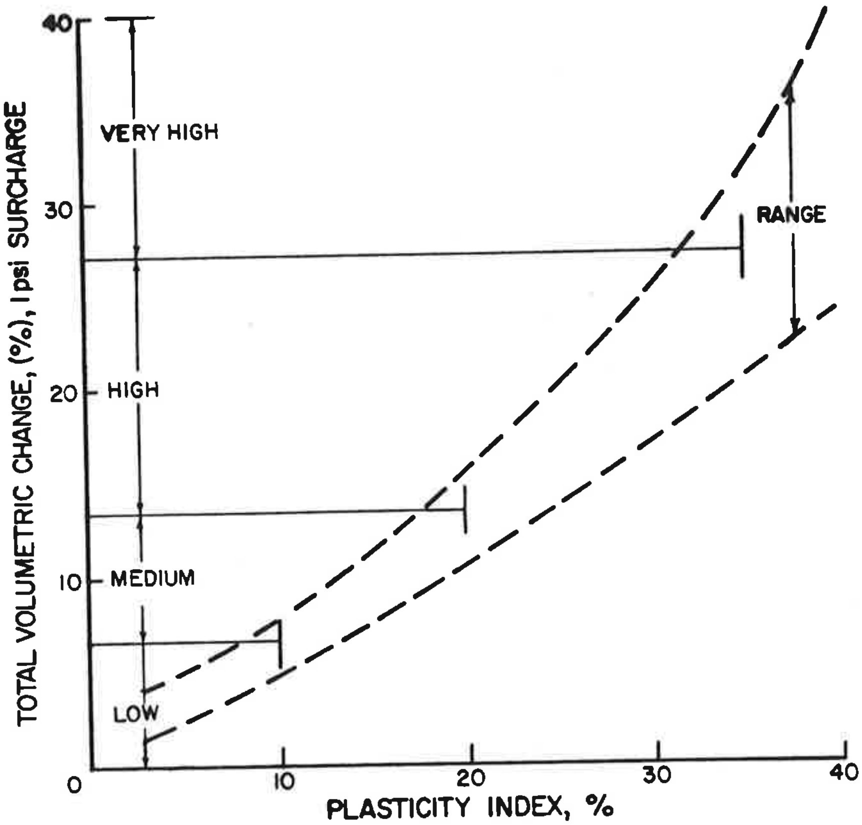
Swell potential represents the capacity of a soil to change volume due to changes in soil moisture content. The most common correlations between swell potential and index properties consider plasticity limits and fine fraction of the soil. A correlation that includes both factors is shown in Figure A-7.1. Two correlations that consider only plasticity are shown in Figure A-7.2.



Reproduced from Pavements on Expansive Clays, p. 33, by Kassiff et al. by permission of Academic Press. Year of first publication: 1969.

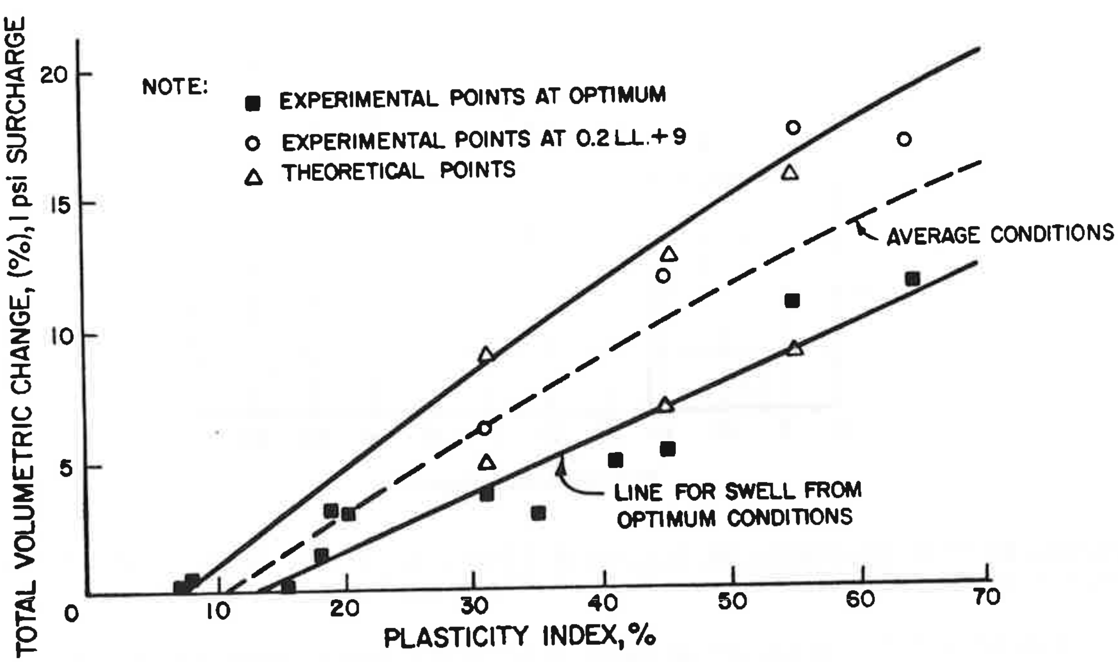
FIGURE A-7.1. RELATION BETWEEN PLASTICITY INDEX, PERCENT SMALLER THAN 1 μm, AND DEGREE OF SWELL

(a)



After Engineering Properties of Expansive Clays, 1956 by Holtz & Gibbs

(b)



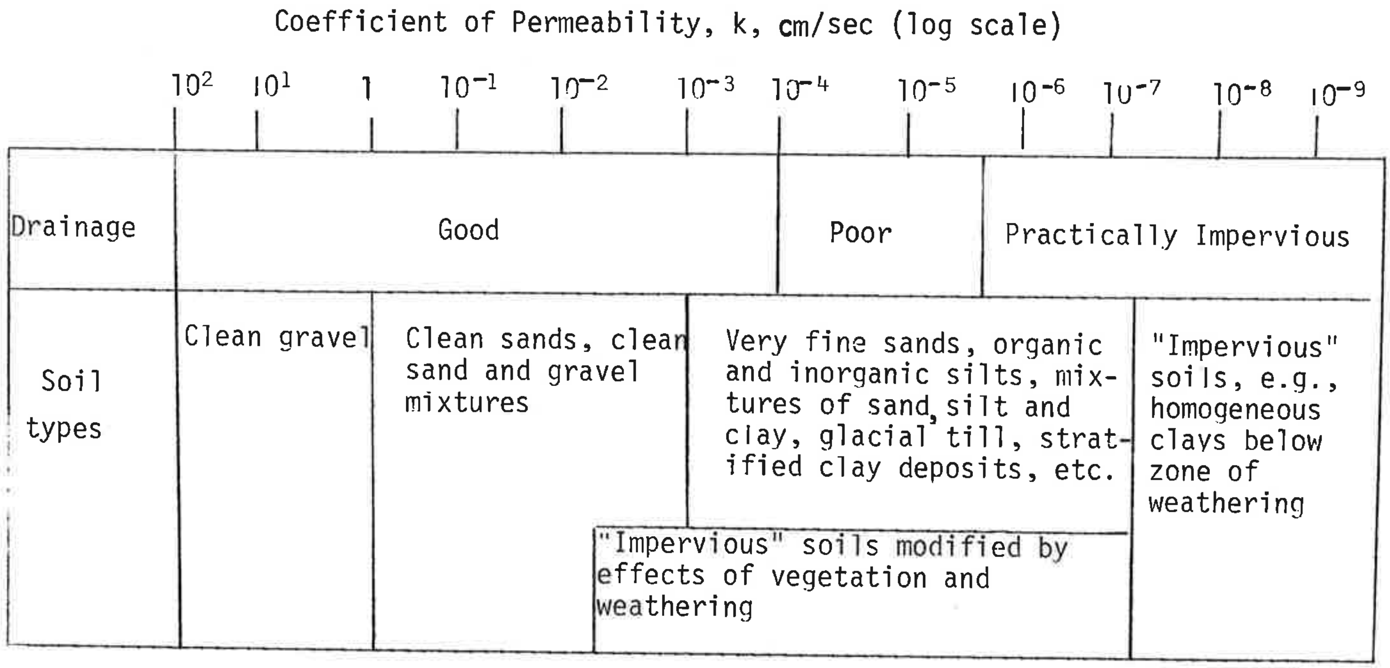
After Interrelationships of Load, Volume, Change, and Layer Thickness of Soils, 1956, by C. Mc Dowell.

FIGURE A-7.2. RELATIONS BETWEEN POTENTIAL VOLUMETRIC SWELL AND PLASTICITY INDEX

A8. PERMEABILITY

A8.1 Typical Values of Soil Permeability

Typical values of permeability based on descriptions of soil grain size are shown below.



From Soil Mechanics in Engineering Practice, second edition, p. 55, by K. Terzaghi and R. Peck, by permission of John Wiley and Sons, Inc. Year of first publication: 1967.

A8.2 Correlations Between Permeability and Grain Size for Granular Soils

Numerous equations have been proposed for approximate evaluation of permeability from soil grain size characteristics and phase relationships. Louden has collected, summarized, and evaluated many of these equations. The Hazen Formula is most widely used. In 1911 Hazen proposed that the permeability of coarse-grained materials was related to the D_{10} grain size by means of the following equation:

$$k = C\ D_{10}^{\ 2}$$

where

k = permeability (cm/sec)

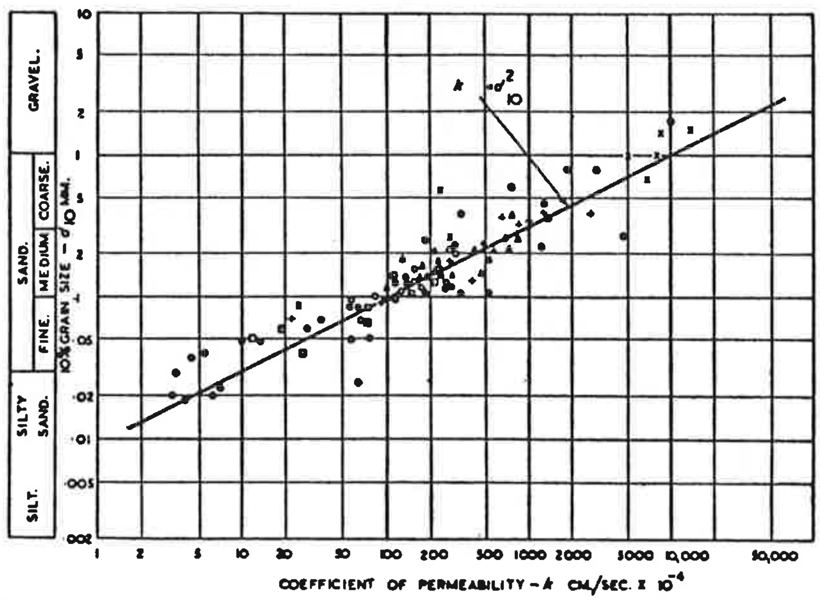
C = factor of proportionality, approximately 100 but varying from 50 to 150

D_{10} = particle diameter coarser than 10 percent of the material by weight, in cm.

Louden's comparison of permeabilities computed by the Hazen formula with permeabilities measured in the laboratory is summarized in Figure A-8.2.

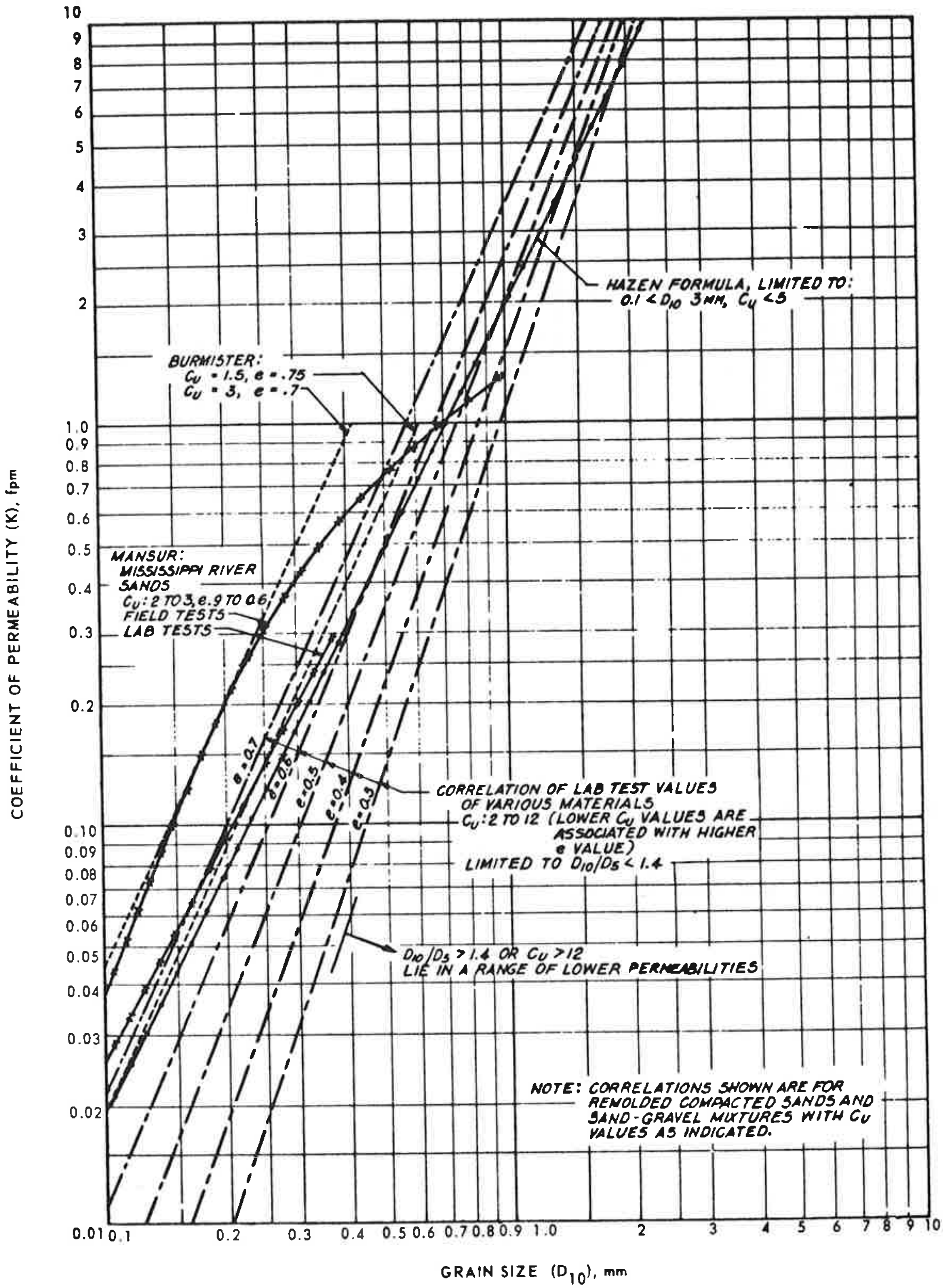
A8.3 Correlations Between Permeability, Grain Size, and Void Ratio for Granular Soils

The Hazen formula is based on measured data on sands with a limited range of grain size and gradational characteristics. Figure A-8.3 uses both the D_{10} size and the void ratio to yield correlations that may compare more favorably with a broader set of data than the Hazen formula. However, as the figure shows, the Hazen formula provides a good approximation to this more-complex correlation.



Reproduced from "The Computation of Permeability from Simple Soil Tests," p. 179, by A.G. Loudon by permission of the Institution of Civil Engineers. Year of first publication: 1953.

FIGURE A-8.2. RELATION BETWEEN D_{10} GRAIN SIZE AND PERMEABILITY

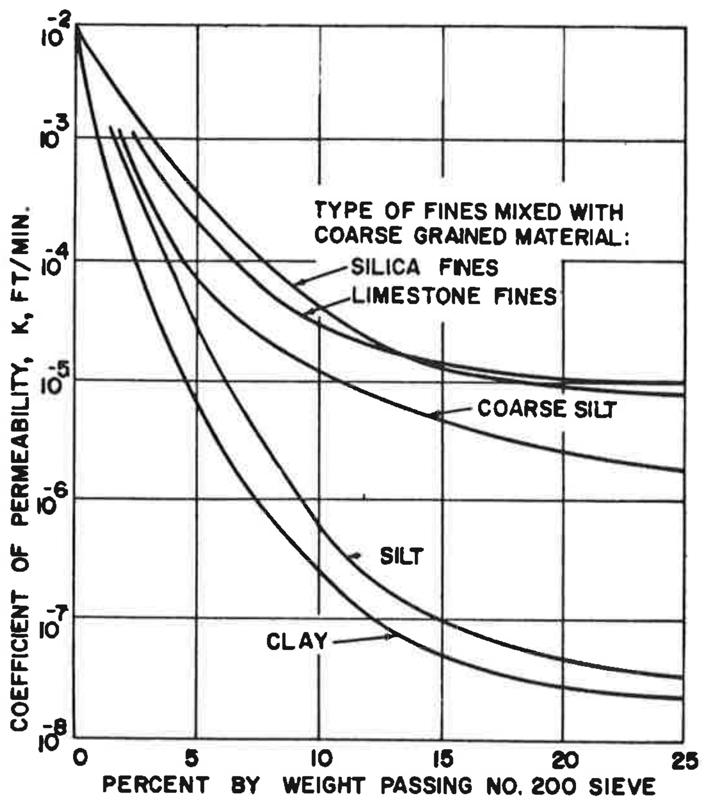


Reproduced from Soil Mechanics, Foundations, and Earth Structures, p. 7-3-10, U.S. NAVFAC Design Manual DM-7. Year of first publication: 1961.

FIGURE A-8.3. PERMEABILITY OF CLEAN SANDS AND SAND-GRAVEL MIXTURES VERSUS D_{10} GRAIN SIZE

A8.4 Permeability of Fine-Grained Soil

Permeability of fine-grained soils are generally orders of magnitude lower than granular soil. The permeability of a mixture of coarse- and fine-grained soil is governed by the fine-grained fraction. Figure A-8.4 shows the influence of the nature and quantity of the fine soil fraction on the resulting permeability of the combined soil.



Reproduced from Soil Mechanics, Foundations, and Earth Structures, p. 7-8-10, U.S. NAVFAC Design Manual DM-7. Year of first publication: 1961.

FIGURE A-8.4. EFFECT OF FINES ON SOIL PERMEABILITY

APPENDIX B

METHODS, EQUIPMENT AND BORING CRITERIA UTILIZED IN PERFORMING GEOTECHNICAL INVESTIGATIONS

The Transportation Research Board sent a questionnaire to all state transportation agencies to ascertain the relative frequency of use of conventional and special methods and equipment currently used in performing geotechnical investigations. Agencies responding are listed in Appendix B. In the summary of results presented hereinafter, every attempt was made to incorporate broad, general descriptions of equipment, with the recognition that local descriptions of identical equipment may vary.

The location, spacing, type, and depths of borings and/or excavations comprising a geotechnical investigation are usually determined on the basis of design considerations and reconnaissance observations for development of maximum subsurface information utilizing a minimum number of borings. Topography, geologic origin of materials, and surface manifestation of soil and rock conditions are of vital concern. As boring and sampling progresses in the field and subsurface conditions become more evident, the locations, spacing, depths, and types of explorations are reviewed and the extent of explorations increased or decreased as considered necessary.

Although it is recognized that flexibility in the subsurface explorations program is necessary and desirable, and reflects good engineering practice, most agencies nevertheless have a more or less standard criterion for conducting geotechnical investigation programs. The following abbreviated summary of criteria is intended to show the approximate extent of investigations completed for design of structures and roadways.

DRILLING

TYPE	NUMBER OF AGENCIES: ROUTINELY USING	INFREQUENTLY USING	NEVER USING
ROTARY:			
Fishtail Bit	19	12	4
Rock Roller Bit	26	6	4
Drag Bit	2	-	-
PERCUSSION:			
Cable Tool	1	3	31
Hammer	12	5	18
AUGER-Mechanical:			
Spoon	3	5	25
Hinged	-	4	28
Disk	-	6	21
Continuous Helical	33	3	1
Hollow Stem	25	6	5
WASH BORING	15	16	7
CONTINUOUS SAMPLE	15	17	1
BORING STABILIZATION:			
Water	29	6	-
Natural Slurry	13	12	10
Artificial Slurry	17	15	6
Air	3	-	-
Freezing	-	1	35
Grouting	-	21	13
Casing	30	7	-

GROUND WATER OBSERVATIONS

FOR DESIGN	NUMBER OF AGENCIES: ROUTINELY USING	INFREQUENTLY USING	NEVER USING
FREE GROUNDWATER LEVEL:			
Uncased Borings	32	4	-
Cased Borings	23	9	2
Well Point			
Well Screen	5	13	15
Porous Tube	5	11	17
Perforated Pipe	1	-	-
PORE PRESSURE:			
Standpipe Piezometer	12	16	9
Closed Hydraulic Piez.	3	12	21
Vibrating Wire Strain Gauge Piezometer	-	5	30
Pneumatic Piezometer	5	11	18
FOR CONSTRUCTION:			
Standpipe Piezometer	12	17	8
Closed Hydraulic Piez.	3	12	19
Vibrating Wire Strain Gauge Piezometer	-	3	31
Pneumatic Piezometer	6	12	18

LABORATORY SOIL TESTING

TEST	NUMBER OF AGENCIES INVESTIGATION STAGE: CORRIDOR STUDY	PRELIMINARY DESIGN	FINAL DESIGN
PARTICLE SIZE ANALYSIS:			
Sieve	10	29	24
Hydrometer	3	21	20
ATTERBERG LIMITS:			
Liquid	10	30	22
Plastic	9	29	19
Shrinkage	1	14	14
MOISTURE CONTENT	8	31	23
ORGANIC CONTENT	4	12	15
SPECIFIC GRAVITY	3	22	21
COMPACTION:			
Standard	1	19	21
Modified	1	9	12
RELATIVE DENSITY	1	9	5
PERMEABILITY:			
Constant Head	-	7	8
Falling Head	-	10	10
CONSOLIDATION:			
Permeability	1	13	10
Hysteresis (Double)	1	8	4
UNCONFINED COMPRESSION	2	23	20
DIRECT SHEAR, DRAINED	1	10	14
TRIAXIAL COMPRESSION:			
Q-Test, (Unconsolidated-Undrained)	2	19	18
R-Test, (Consolidated-Undrained)	1	12	14
$\bar{R}$-Test, (Consolidated-Undrained w/ Pore Pressure Measurements)	1	10	10
S-Test, (Consolidated-Drained)	1	14	9
MINIATURE VANE:			
Laboratory Vane	-	4	3
Manual Vane	-	1	3

CONSTRUCTION MONITORING INSTRUMENTATION

INSTRUMENT	NUMBER OF AGENCIES: ROUTINELY USING	INFREQUENTLY USING	NEVER USING
INCLINOMETERS	11	16	9
SHEAR STRIP	-	-	34
SETTLEMENT PLATES	14	17	5
PIEZOMETERS:			
Standpipe	13	17	6
Closed Hydraulic	3	12	20
Vibrating Wire Strain Gauge	1	1	29
Pneumatic	7	8	18
DISPLACEMENT STAKES	12	15	9
TELLTALES	4	5	23
EARTH PRESSURE CELLS	-	10	25
STRAIN GAUGES	1	6	29
ROCK BOLT LOAD CELLS	-	2	30

SAMPLING

TYPE	NUMBER OF AGENCIES ROUTINELY USING	INFREQUENTLY USING	NEVER USING
PENETRATION TESTS:			
Split Barrel	33	2	2
Split Barrel w/liner	5	17	14
Large Diameter Split Barrel w/liner	1	14	21
Solid Barrel	1	6	22
Solid Point	1	-	-
THIN WALL TUBE:			
Shelby Tube	31	6	-
Fixed Piston	7	11	16
Pitcher Sampler	2	7	26
WASH	10	15	11
RETRACTABLE PLUG	6	7	20
PEAT SAMPLER	7	6	21
ROCK CORING:			
Size EX	2	1	4
Size AX	10	3	-
Size BX	9	1	1
Size NX	20	1	-
CORE BARRELS:			
Single Tube	10	12	11
Double Tube	30	4	2
Wire Line	2	4	26
Denison	1	4	-
CORE BITS:			
Diamond	34	2	-
Hardened Surface	18	23	2
HOLE ORIENTATION:			
Vertical	32	3	-
Horizontal	-	1	-
Angle	4	13	18
BORE HOLE CAMERA	1	3	32
ACCESSIBLE EXPLORATIONS:			
Test Pits	9	17	11
Test Trenches	4	24	7
Caissons	-	14	-
Accessible Borings	17	4	10

FIELD TESTS

	NUMBER OF AGENCIES ROUTINELY USING	INFREQUENTLY USING	NEVER USING
DYNAMIC:			
Standard Penetration Test	34	1	1
Cone Penetrometer	5	8	23
Driven Probe	5	11	19
Driven Casing	11	10	13
Drive Rod	1	-	-
STATIC:			
Cone Penetrometer	2	5	31
Field CBR	2	8	25
Plate Bearing Test	1	20	13
Lateral Bearing Test	1	-	-
IN-PLACE-VANE	8	18	15
PRESSUREMETER	1	4	20
PERMEABILITY:			
Falling Head	6	18	12
Pumped-In	-	11	23
Pumped Well	-	1	-
IN-PLACE SOIL DENSITY			
Sand Cone	1	5	9
Rubber Ballon	-	4	7
Nuclear	2	4	9
Drive Cylinder	1	2	6
Shelby Tube	-	1	2
Ring Sampler	-	1	1

GEOPHYSICAL

	NUMBER OF AGENCIES ROUTINELY USING	INFREQUENTLY USING	NEVER USING
ELECTRICAL:			
Resistivity	11	15	10
RADIATION:			
Natural Gamma Ray	3	2	29
Gamma-Gamma Ray	-	2	28
Neutron	3	2	28
ACOUSTICAL			
Velocity (porosity)	-	1	35
Amplitude (Location of fracture zones)	-	1	35
SEISMIC			
Standard Refraction Survey	11	10	13
Sonar Continuous Seismic Profiling			
Boomer Probe	-	1	35
Pinger Probe	-	1	34
Standard Reflection Survey	-	7	29

ABBREVIATED SUMMARY

ROADWAY BORING CRITERIA

SITUATION	BORING SPACING	BORING LOCATION	MINIMUM DEPTH	BORING TYPE	MINIMUM NUMBER BORING OR CROSS-SECTIONS
EMBANKMENTS - Roadway:	1 per fill to 1 each 400-500 ft	Centerline or Ditchline	(Emb Ht +10') (2/3 Emb Ht.) (Firm Material) (10' into firm material)	Auger, Undist. Samples. Backhoe	1 to 5
CUTS Roadway:	25' to 400'	Centerline or Ditchline	(2' to 10' below grad) (Firm Material)	Auger, Backhoe	1 to 5
CUT-FILL	(Same as above)				
SPECIAL INVESTIGATIONS Cut Slope Stability	(25' to 300') (1 per cut)	Midpoint Top & Toe Slope	(5' to 70') (variable) (10' into stable soil)	Auger, Continuous Samples	2 to 3
Emb.-Found. Stability	(approximately same as above)				
Settlement Studies	(50' to 300')	Centerline	Refusal or Hard Layer	Auger, *SPT, Continuous Samples	
CONSTRUCTION MATERIAL Borrow Sources	50' to 500'	Grid	Variable	Auger, to Pit Excavations	Varies

*SPT - Standard Penetration Test - ASTM D 1586-67

APPENDIX C

RECOMMENDED PROCEDURES FOR BALLAST INDEX TESTS

Most of the recommended procedures for index tests of railroad ballast, listed in Table 3.2, are based on procedures published in Parts 14 and 19 of the 1979 Annual Book of ASTM Standards; Volume 812 of the British Standards Institute's Standard Tests (1975); and Suggested Methods of the International Society for Rock Mechanics (1972 and 1974). This appendix contains specific details for applying standard testing methods to ballast. In addition, more complete procedures are provided for tests that are not described in the above standards publications.

Sampling is as important as testing, and should be done in accordance with ASTM Standard Method D75. A sample size should weigh at least 100 kg. Research has shown that representative samples can be gathered by collecting many small specimens from several locations and then combining them into a single sample. After sampling is completed, the following procedures should be used to test the specimens.

C1. PETROGRAPHIC ANALYSIS

Petrographic analysis should be performed in accordance with ASTM Recommended Practice C295 by an experienced petrographer. The petrographer should use whatever tools are necessary to evaluate the ballast, including hand examination of specimens, polished sections, thin sections, x-ray diffraction, chemical analysis, and powdered sample examination. He or she should be familiar with the performance requirements for railroad ballast and the type of evaluation required, including an assessment of the following ballast material properties:

- An accurate determination of geologic rock classification and the common type of ballast rock. The rock classification system that should be used is outlined in ASTM Standard C294. In addition, ballast should be classified into rock groups historically used by the railroad industry, such as granite, traprock, limestone, slag, and gravel.
- A description and quantitative percentage estimate of mineral constituents based on examination of hand specimens, sections, and Los Angeles abrasion fines. Predominant minerals and any minor minerals that may significantly influence ballast performance -- such as chlorites, sulphides, or other easily weathered, soft, or easily fractured grains or cement -- should be noted.
- An evaluation of fines from the Los Angeles abrasion test including an estimate of the rock flour fraction and grain chips fraction. A predominance of rock flour indicates ballast with soft minerals that may abrade to plastic, low-permeability fines. A predominance of rock chips may indicate ballast with hard minerals that will retain a high permeability even if the ballast disintegrates.
- A description of additional features that may influence physical properties of ballast material, such as grain size (which may affect toughness), porosity (freeze-thaw resistance), grain orientation (preferred cleavage direction), degree of sedimentary induration (toughness),

mineral hardness (abrasion resistance), and mineral chemical activity (weatherability).

- An examination of a sufficient number (i.e., 5 to 50, depending on the character of the rock) of specimens to ensure that a representative sample is used. If the ballast source is a layered deposit, such as sedimentary limestone or banded gneiss, sampling should be conducted by a geologist in the field so that specimens can be taken from several beds.

C2. BULK SPECIFIC GRAVITY, WATER ABSORPTION

In conducting this test, select a sample that represents the condition of the material as it will be delivered to the track. Test in accordance with ASTM Standard Method C127, except do not oven-dry the sample in advance, as stated in Paragraph 5.1. Prior to testing, soak the sample in water for 24 hours, and compute only bulk specific gravity and absorption.

C3. GRAIN SPECIFIC GRAVITY

Grain specific gravity should be determined in accordance with the pulverization method recommended by the International Society for Rock Mechanics (ISRM) in their Suggested Methods, 1972. As part of this test procedure, a representative sample of ballast is pulverized to a fine size so that only single-grain particles remain. The grain specific gravity is determined by the volumetric method in accordance with ASTM Standard Method D854.

C4. TOTAL POROSITY

Total porosity of samples can be calculated by using the following equation:

$$n = 1 - B/G_s$$

where n = total porosity
B = bulk specific gravity
G_s = grain specific gravity

C5. DEGREE OF SATURATION

Degree of saturation is defined as the volume of voids in a rock filled with water, divided by the total volume of voids (usually expressed as a percentage). The degree of saturation can be calculated by using the following equation:

$$S = \frac{A}{\frac{1}{B} - \frac{1}{G_s}} \times 100$$

where

S = degree of saturation (in percent)
A = absorption (in percent)
B = bulk specific gravity
G_s = grain specific gravity

C6. LOS ANGELES ABRASION

The Los Angeles abrasion test is generally executed in accordance with ASTM Standard Method C535. When conducting this test, a sample grading of 2 or 3*, whichever will incorporate the larger fraction of the total ballast sample, is selected. Abrasion loss should be measured after 200 cycles, using the dry sieving method called for in Note 5 of ASTM Standard Method C535 and measured by wet sieving methods after 1,000 cycles.

A second test should be conducted with a matching sample, adding 5 kg of distilled water to the 10 kg of the sample in the drum. The door of the drum should be sealed with an appropriate gasket to prevent the loss of water or the sample. The wet sample should be sieved only after 1,000 cycles, using the wet sieving method.

C7. POINT LOAD COMPRESSIVE STRENGTH

The point load compressive strength test should be carried out in accordance with the procedure defined by ISRM's Suggested Method for Determining Shear Strength (1974) or by Brock and Franklin in "The Point-Load Strength Test" (1972). When conducting this test, select equi-dimensional ballast pieces from 30 mm to 70 mm in approximate diameter, with a ratio of longest to shortest dimension no greater than 1.4. It's advised to air dry samples before testing and to test at least 25 pieces. The load platens should be two 60° cones tipped with 5-mm radius spheres. The results should be reported as the mean and standard deviation of the point-load strength index corrected to 50 mm, as described in the ISRM test procedure.

C8. MILL ABRASION

The mill abrasion test is best described in an unpublished report by Raymond, Material Specification for Railroad Ballast. The following test procedures are slight variations of the procedures recommended by Raymond:

a. Select a sample made up of two fractions, with each fraction weighing 1,500± 10 g of the same sized groups used in the Los Angeles abrasion test. (The total sample weight should be 3,000± 15 g.) Oven-dry the sample and weigh it to the nearest 0.01 g.

b. Place the aggregate sample in a 5-litre, 230-mm in outside diameter, porcelain ball mill pot. Add 3 litres of distilled water to the sample.

---

*Grading 2:	5 kg - 50 mm to 38.1 mm
	5 kg - 38.1 mm to 25 mm
Grading 3:	5 kg - 38.1 mm to 25 mm
	5 kg - 25 mm to 19 mm

c. Seal the mill pot and rotate it 33 revolutions per minute for 10,000± 100 revolutions.

d. Then transfer the sample to a No. 200 sieve (i.e., 0.075 mm) and wash the sample to remove the -No. 200 mesh fines, in accordance with ASTM Standard Method C117.

e. Oven dry the remaining sample to constant weight.

f. Re-weigh the sample to determine abrasion loss. Express the mill abrasion loss as the weight loss divided by initial dry sample weight (as a percentage).

C9. SULPHATE SOUNDNESS

The sulphate soundness test should be conducted in accordance with the procedure defined in ASTM Standard Method C88. The magnesium sulphate solution called for in Paragraph 4.12 of the ASTM procedure should be used. Coarse aggregate fractions selected in accordance with paragraph 5.2 of this procedure should then be tested. The following fractions should each be tested as may be available from the total sample:

19.0 mm to 9.5 mm
38.1 mm to 19.0 mm
63.0 mm to 38.1 mm

Wash the aggregate fractions in clean water, and then dry and sieve them to determine loss after five cycles and ten cycles. Both loss values should be reported.

C10. REFERENCE DENSITY

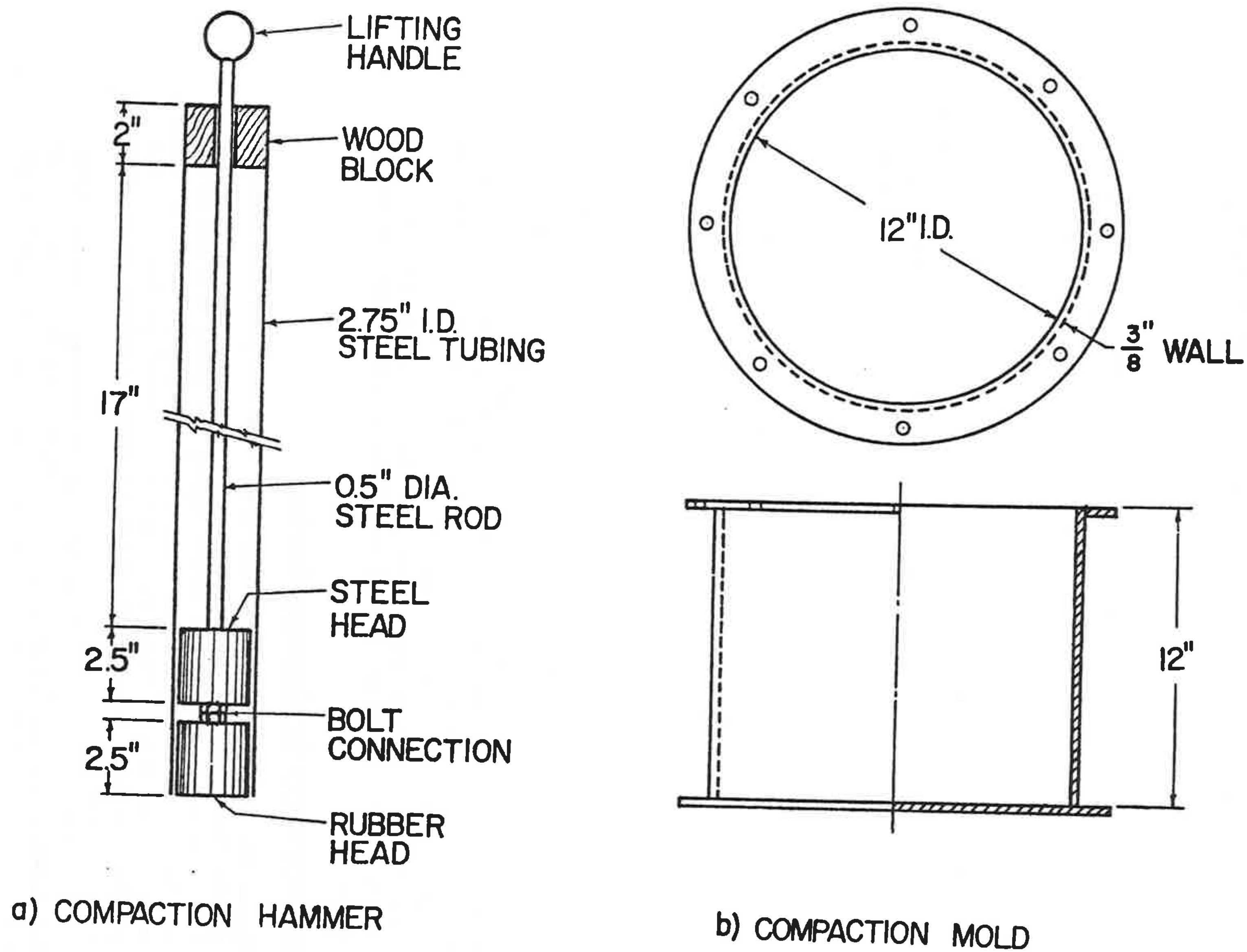
The reference density test should be conducted in accordance with the procedure suggested by Selig et al. in Field Methods for Ballast Physical State Measurements. Results should be expressed in terms of density (Mg/m^3) and bulk porosity. Density and bulk porosity should be reported for uncompacted and ultimate compacted conditions. A recommended procedure adopted from the above-referenced report is included in the following material.

C10.1 Apparatus Used in Reference Density Test

The following tools are used to test reference density of ballast materials:

Compaction Hammer

Compaction is accomplished with a manually operated impact hammer (See Figure C-1a) having a 70-mm diameter circular face, tipped with a rubber cylinder, with a mass of 3.5 kg. The hammer is equipped with a guide sleeve to control the height of drop to a free fall of 0.43 m above the surface of the ballast sample. The guide sleeve has at least four vent holes, which are not smaller than 9.5-mm in diameter and are spaced 90 degrees apart and 19 mm from each end, and provides sufficient clearance so that the free fall of the hammer shaft and head will not be restricted.



Reproduced from Field Methods for Ballast Physical State Measurements, p. 149, by E.T. Selig et al. Year of first publication: 1977.

FIGURE C-1. REFERENCE DENSITY APPARATUS

Compaction Mold

The compaction mold has an internal diameter of 0.305 m and a height of 0.305 m (See Figure C-1b). The volume is 0.0222 cubic metres.

Rubber Membrane

A membrane is used to determine volume by the water replacement method. The membrane, which should be at least 0.6 m square, should be very thin and should conform to the top surface of the ballast. A 0.2 mm thick rubber sheet is recommended.

Supporting Equipment

The supporting equipment includes a balance or scale of at least 20 kg capacity with a sensitivity of 0.1 g; a container of at least 35 litres capacity for water; a graduated cylinder with a capacity of approximately 1 litre; and a ruler, sample pan, and scoop.

C10.2 Test Procedures for Uncompacted Samples

a. Select a representative sample of about 0.03 m^3 of ballast representative of the ballast to be delivered to the track in the field. Oven-dry the sample at 110°±5°C to a constant weight and let cool.

b. Mix the prepared sample thoroughly, using a scoop.

c. Gently pour the ballast loosely into the container in a uniform layer from a height less than 50 mm. Move the scoop in a spiral motion from the outside of the container toward the center to form a uniform layer without particle segregation. Continue this process until the container is approximately 80-percent full.

d. Level the surface of the ballast in the container by gently filling any large voids and by removing any oversized particles.

e. Measure the volume of the ballast in the container, using the procedures described below under "Sample Volume Determination Procedure,"

f. Weigh the ballast in the container.

g. Perform at least two independent fillings and measurements. The average of all tests completed is used as the bulk density of the uncompacted sample.

C10.3 Test Procedures for Compacted Samples

a. Reuse the uncompacted sample obtained from the previous procedure. Mix the sample thoroughly, using a scoop.

b. Place the ballast into the sample container using a scoop with a drop height less than 50 mm. The ballast should be placed in three layers, each 90-mm to 110-mm thick. Move the scoop in a spiral motion from the outside of the container toward the center to form a uniform layer without particle segregation.

c. Compact the loose layer of ballast by delivering 10 blows from the impact hammer. For each blow, allow the hammer to fall freely from a height of 0.43 m and evenly distribute the blows over the surface of the sample.

d. Repeat the second and third steps for the next two layers. When completed, the sample should be between 225 mm and 250 mm high.

e. Level the surface of the ballast of the container by filling any large voids and by removing any oversized particles.

f. Measure the volume of the ballast in the container, using the procedures described below under "Sample Volume Determination Procedure."

g. Weigh the ballast in the container.

h. At least two independent tests should be made for each compactive effort (i.e., blows per layer). The average of all tests performed is used as the bulk density of the compacted ballast.

i. Repeat the compacted sample procedure using 20 and 40 compaction blows per layer.

C10.4 Sample Volume Determination Procedure -- Water Replacement

a. Place the compaction mold on a level surface.

b. Lay a rubber membrane loosely over the top surface of the ballast sample so that it's in as close contact as possible with the inside of the mold and the ballast surface.

c. Fill the depression in the membrane with water to within 25 mm to 50 mm from the top of the mold.

d. Measure the volume of water added and the distance from the top edge of the mold to the water surface in at least four positions that are equally spaced apart along the circumference of the mold, using a straight edge and calibrated probe.

e. Based on these measurements, determine the unfilled volume of the mold between the top edge of the container and the ballast surface. Calculate the volume of the ballast sample by subtracting the unfilled volume from the total volume of the container.

C10.5 Calculations Used in Reference Density Test

The following two calculations are used in conducting reference density tests.

Compactive Effort and Density

Calculate the compactive effort and bulk density of the compacted ballast sample for each trial, using the following equations:

$$E = \frac{3M_r gDN}{V_c}$$

$$\gamma wr = \frac{M_s}{Vcwr}$$

$$n = 1 - \frac{\gamma wr}{B\gamma_w} \times 100$$

where

E = compacting effort (J/m^3)

γwr = unit weight of a ballast sample obtained from the water replacement method (Mg/m^3)

V_{cwr} = volume of a ballast sample determined from using the water replacement method
M_s = mass of sample (kg)
n = bulk porosity (%)
B = bulk specific gravity of particles
γ_w = unit weight of water (1 Mg/m^3)
M = mass of hammer (kg)
g = acceleration of gravity (9.807 m/sec^2)
D = free fall distance of hammer (m)
N = number of blows per layer

Ultimate Density

The values of bulk density and compactive effort from the reference density test are plotted in the form shown in Figure C-2. The data are assumed to fit the hyperbolic form of Figure C-2a, which may be plotted in linear form using the coordinates shown in Figure C-2b. A straight line is fit through the points in Figure C-2b, either by eye or by least-squares curve-fitting methods. Density is calculated by the relationship

$$\gamma_{ult} = \frac{1}{b_t} + \gamma_o$$

where

b_t = slope of the line in Figure C-2b
γ_o = uncompacted unit weight when E = 0

C11. FLAKINESS, ELONGATION INDICES

The flakiness and elongation index tests should be executed in accordance with British Standards Institute Tests[1]. Preferably, the person conducting the test should use special sieves and gauges prescribed by the standard procedures or the equivalent U.S. standard sieves listed in Table C-1.

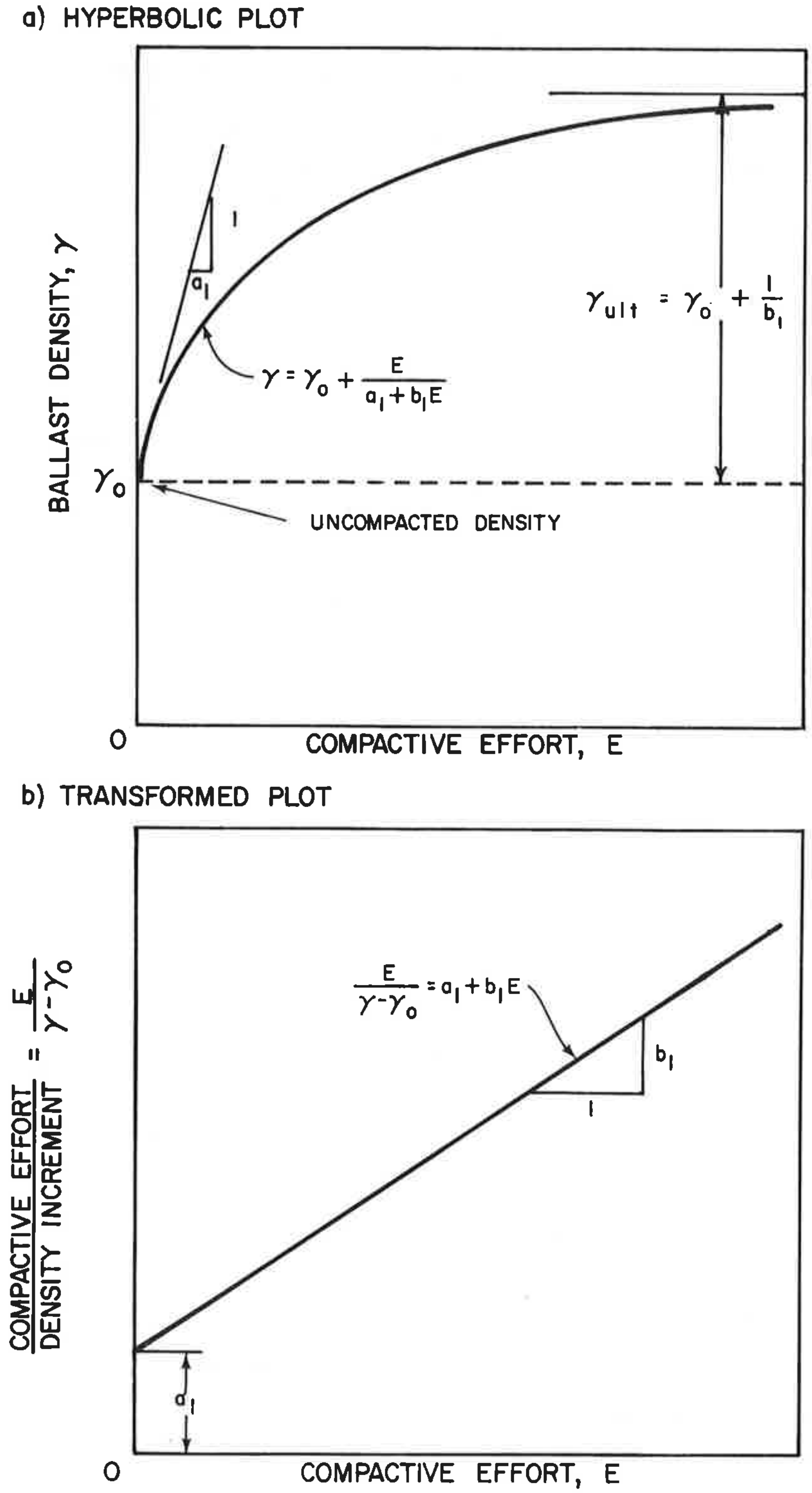
If slotted sieves and elongation gauges are not readily available, determine flaky and elongated particles by using size fractions that are separated by standard sieving procedures and long- and short-axis dimensions determined by caliper or other direct measurements.

The general procedures for the flakiness and elongation index tests follow:

C11.1 Flakiness Index

- Select a sample amount that will yield at least 200 pieces of each size fraction constituting more than 15 percent of the total sample, and at least 100 pieces of each size fraction comprising 5 percent to 15 percent.

(1)British Standards Institute, Standard Tests, Vol. 812, 1975, pp. 32-33.



From "Field Methods for Ballast Physical State Measurements," p. 155, by E.T. Selig et al. Year of first publication: 1982.

FIGURE C-2. REPRESENTATION OF RELATIONSHIP BETWEEN DENSITY AND COMPACTION EFFORT

- Dry the sample in an oven at 110°±5°C.
- Separate the sample into size fractions by sieving in accordance with ASTM Standard Method C136. Use size fractions specified in the British Standards Institute procedure or sizes listed in Table C-1.
- Gauge each size fraction using thickness gauges or slotted sieves. Weigh flaky pieces and the total sample to an accuracy of at least 0.5 percent.
- The flakiness index should be expressed as a percentage weight of all flaky pieces divided by the total sample weight for all size fractions combined.

C11.2 Elongation Index

- Follow the first through third steps of the Flakiness Index procedure.
- Gauge each size fraction using length gauges. Weigh elongated pieces and the total sample to an accuracy of at least 0.5 percent.
- The elongation index should be expressed as a percentage weight of elongated particles divided by the total sample weight for all size fractions combined.

TABLE C-1. DIMENSIONS OF SIEVES, THICKNESS, AND ELONGATION GAUGES USED FOR FLAKINESS AND ELONGATION INDEX TESTS

Size of Aggregate							
Passing U.S. Sieve		Retained U.S. Sieve		Thickness Gauge*		Length Gauge+	
in.	mm	in.	mm	in.	mm	in.	mm
2 1/2	63.0	2	50.0	1.33	33.9	4.00	101.7
2	50.0	1 1/2	38.1	1.04	26.4	3.12	79.3
1 1/2	38.1	1	25.0	0.75	18.9	2.23	56.8
1	25.0	3/4	19.0	0.52	13.2	1.56	39.6
3/4	19.0	1/2	12.5	0.37	9.4	1.12	28.4

* 0.6 times the mean sieve size
+ 1.8 times the mean sieve size

NOTE: Gauge sizes to be maintained at ± 0.01 inch/± 0.2 mm

C12. SIEVE ANALYSIS

The person performing the sieve analysis test should follow ASTM Standard Method C136. Results should be presented as a plot of a percentage weight passing through versus sieve dimension plotted to logarithmic scale as shown in Figure 3.1. The coefficient of uniformity $C_u = D_{60}/D_{10}$ where D_{60} and D_{10} are the sieve sizes passing 60 percent and 10 percent of the total sample, respectively, should be reported as well as the gradation modulus ($\bar{A}$). Calculate $\bar{A}$ as the weighted sum of the individual A_i as follows:

$$\bar{A} = \frac{1}{100} \Sigma\, p_i\, \bar{A}_i$$

where

p_i = the percentage of the sample retained between sieves of size d_1 and d_2 (in mm), Σ indicates summation,

$$\bar{A}_i = \frac{\log 54.8 - \log \bar{d}_i}{\log 2} = 3.32 \log (54.8/\bar{d}_i)$$

$$\bar{d}_i = \frac{d_1 - d_2}{\log_e(d_1/d_2)}$$

Alternatively, the gradation modulus may be calculated simply as

$$\bar{A} - \frac{1}{100} \Sigma\, P_i$$

where

P_i = the percent of sample passing each of the following U.S. standard sieves: 1 1/2 inch, 3/4 inch, 3/8 inch, No. 4, No. 4, No. 8, No. 16, No. 30, No. 50, No. 100, and No. 200.

C13. CRUSHING VALUE

The crushing value test is performed in accordance with the previously mentioned British Standards Institute procedure(1). The person conducting the test should use a sample that passes a 12.5-mm sieve and is retained on a 9.5-mm sieve. The sample should be air dried before testing. For sieving fines, U.S. No. 8 sieve (2.36 mm) may be substituted for British No. 7 sieve (2.40 mm). The plunger should be loaded to 40 tons in 10 minutes. Maintain the load for less than one minute and release the load in less than two minutes following maintenance of the maximum load.

C14. CEMENTING VALUE TEST

There is no standard procedure to determine the cementing value of aggregates. The procedure adopted by the Pennsylvania Railroad and Conrail is the most widely used but some of the test details are not well defined; this leads to variability in results. Research into developing a cementing

(1) British Standards Institute, Standard Tests, Vol. 812, 1975, pp. 75-79.

value test procedure that produces consistent results is needed before the results of the test can become a reliable indicator of ballast performance. A tentative recommended procedure based on the ConRail procedure follows.

a. Secure a dry sample of fines derived from the Los Angeles abrasion (LAA) test run without water after 1,000 revolutions.

b. Screen fines through a No. 4, No. 40, and No. 100 U.S. Standard sieves. A sample of at least 200g minus No. 100 material is required. If insufficient fine material is secured, return all material coarser than No. 100 sieve to LAA drum and rotate sufficient additional turns to produce the required quantity of minus No. 100 fines.

c. Take approximate 25g of the minus No. 100 fines and determine the plastic limit of the material in accordance with ASTM Method D424.

d. After determining the plastic limit, add sufficient water to the remaining sample to bring the sample to the plastic limit. Thoroughly knead the sample for 5 minutes to achieve an homogeneous dough. Some extra water may have to be added to account for evaporation during kneading.

e. Cure the dough for 2 hours in an air-tight container at 21±3°C.

f. Mold dough into at least 5 cylindrical molds 25.4mm diameter and 25.4 mm high (1 inch by 1 inch). Apply a molding pressure of 13 MPa (1885 psi).

g. Weigh each mold plus sample immediately after molding.

h. Dry samples for 20±2 hours in room at 21±3°C; oven dry for 4±0.5 hours in an oven at 110±5°C (230±9°F); cool in a dessicator for 30±10 minutes at 21±3°C.

i. Weigh each mold plus sample, remove from mold and weigh each sample. Determine as-molded water content. Molded water content should be within 2 percent of previously determined plastic limit.

j. If necessary, plane ends of cylinders with No. 400 emery paper to provide smooth bearing surface.

k. Test compressive strength of cylinder using a load frame equipped with a suitable spherical bearing to provide a uniform compressive load. Apply load at a rate of 2600±400N (600±100 pounds) per minute.

l. Report cementing value in MPa as the average of at least 5 compression tests.

APPENDIX D

CLASSIFIED BIBLIOGRAPHY OF SUBGRADE STABILIZATION

D.1 DRAINAGE

Calhoun, C. C. and W. E. Strohm, "Performance of Plastic Filter Cloths as Replacement for Granular Filter Material," Highway Research Record, No. 373, 1971, pp. 74-85.

Cedergren, H. R. and W. R. Lovering, "The Economics and Practicability of Layered Drains for Roadbeds," Highway Research Record, No. 215, 1968, pp. 1-7.

Machii, K., "Mud Pumping on Tracks--Present State and Countermeasures," Japanese Railway Engineering, Vol. 17, No. 4, 1978, pp. 20-21.

Posey, C. J., "Terzaghi's inverted filter - an Important Tool for the Civil Engineer," Civil Engineering, ASCE, Vol. 46, No. 4, 1976, pp.78-79.

Smith, T. W., and G. V. Stafford, "Horizontal Drains on California Highways," Journal of the Soil Mechanics and Foundations Division, ASCE, July 1957.

Steinberg, M. L., "Interceptor Drains in Heavy-Clay Soils," Transportation Engineering Journal, ASCE, Vol. 96, No. TE1, 1970, pp. 1-9.

Zaruba, O. and J. Simek, "Postglacial Landslide Reactivated by Excavation of Railway-Cutting," Proceedings of the First Congress of the International Association of Engineering Geologists, Paris, 1970, Vol. 1, pp. 74-80.

D.2 IN-PLACE MODIFICATION

Grouting

American Railway Engineering Association, "Roadbed Stabilization," Proceedings, Vol. 53, AREA, 1952, pp. 736-742.

American Railway Engineering Association, "Costs and Maintenance Data on Grouting Projects," Proceedings, Vol. 54, Bulletin, No. 507, AREA, February 1953, p. 1136.

Brown, D. R. and J. Warner, Journal of the Soil Mechanics and Foundation Division, ASCE, Vol. 99, No. SM8, August, 1973, pp. 589-601.

Committee on Grouting, "Chemical Grouting Bibliography," Journal of the Soil Mechanics and Foundation Division, ASCE, Vol. 92, No. SM6, November, 1966, pp. 39-66.

Farris, J. B., "Roadbed Stabilization: Various Methods," Proceedings, Vol. 76, Bulletin, No. 651, AREA, 1975, pp. 419-431.

Hardy, R. M., "Prevention of Frost Heaving by Injection of Spent Sulphite Liquor," Proceedings of the 3rd International Conference on Soil Mechanics and Foundation Engineering, Vol. II, 1953, pp. 103-106.

Herdon, J. and T. Lenahan, "Grouting in Soils, A State-of-the-Art Report," Vol. 1, U.S. Department of Transportation, Federal Highway Administration, Washington, D.C., FHWA-RD-76-26, June 1976, 303 pp.

Herdon J. and T. Lenahan, "Grouting in Soils, Design and Operations Manual," Vol. 2, U.S. Department of Transportation, Federal Highway Administration, Washington, D.C., FHWA-RD-76-27, June 1976, 101 pp.

Hilton, I.C., "Grout Selections: A New Classification System," Civil Engineering and Public Works Review, September 1967, pp. 993-995.

Plaisted, A. C., "The Application of Chemical Grouts to Ground Consolidation," Ground Engineering, Vol. 7, No. 4, 1974, pp. 42-44.

Smith, R. and R. B. Peck, "Stabilization by Pressure Grouting on American Railroads," Geotechnique, Vol. V, No. 3, September 1955, p. 243.

Smith, R. "Stabilization Treatment of Sliding Fills by Pressure Grouting," Proceedings, Vol. 49, AREA, 1948, pp. 612-613.

Spang, J., "Planumschutzschichten unter Eisenbahngleisen" ("Layers protecting the track formation"), Archiv fur Eisenbahntechnik, November 1965, pp. 181-211. (translation No. 667 of British Railways Research Department).

Lime Slurry Pressure Injection, Other Lime Treatments

Blacklock, J., ed., Proceedings of the Roadbed Stabilization Lime Injection Conference, Little Rock, Arkansas, November, 1975, 295 pp. (PB-251681).

Blacklock, J. R., "Evaluation of Railroad Lime Slurry Stabilization," U.S. Department of Transportation, Federal Railroad Administration, Washington, D.C., June 1978, FRA/ORD-78/09, 190 pp. (PB-282998)*

---

*Numbers in parenthesis at the end of an entry indicate the NTIS order number for the publication.

Blacklock, J.R. and C.H. Lawson (as revised by R.H. Ledbetter) "Handbook for Railroad Track Stabilization Using Lime Slurry Pressure Injection," U.S. Department of Transportation, Federal Railroad Administration, Washington, DC, April 1979 (revised), FRA/ORD-77/30 (PB272 721).

Broms, B. B. and P. Boman, "Stabilization of Soil with Lime Columns," Ground Engineering, Vol. 12, No. 4, May 1979, p. 23-32.

Farris, J. B., "Roadbed Stabilization - Various Methods" Proceedings, Vol. 76, Bulletin, No. 651, AREA, 1975, pp. 419-431.

Farris, J. B., "Drill-Lime Treatment of Shallow Railway Subgrade Failures in Expansive Clays," Proceedings, Vol. 72, Bulletin, No. 626, AREA, Feb. 1970, pp. 574-579.

Lundry, H. L. and B. J. Greenfield, "Evaluation of Deep In-Situ Soil Stabilization by High-Pressure Lime Slurry Injection," Highway Research Record, No. 235, 1968, pp. 27-35.

"Southern Injects Lime Slurry to Stabilize Soft Track," Railway Track and Structures, Vol. 68, No. 9, September, 1972, pp. 36-37.

Spang, J., "Planumschutzschichten unter Eisenbahngleisen" ("Layers protecting the track formation"), Archiv fur Eisenbahntechnik, November 1965, pp. 181-211. (Translation No. 667 of British Railways Research Department).

Thompson, M. R. and Q. L. Robnett, "Pressure-Injected lime for Treatment of Swelling Soils," Transportation Research Record, No. 568, pp. 24-34.

Deep Densification

Anderson, R. D., "New Method for Deep Sand Vibratory Compaction," Journal of the Construction Division, ASCE, Vol. 100, No. CO1, March 1974, pp. 79-95.

Ground Treatment by Deep Densification, London: Institution of Civil Engineers, 1976, 154 pp.

Leonards, G. A., W. A. Cutter, and R. D. Holtz, "Dynamic Compaction of Granular Soils," Journal of the Geotechnical Engineering Division, ASCE, Vol. 106, No. GT1, Proceedings Paper 15144, Janurary 1980, pp. 35-44.

Litvinov, I. M., "Osnovaniya, Fundamenty i Mekhanika Grunntov-Sbornik," ("Field Tests of Deep Compaction of Sagging Soils by Presoaking and Explosive Energy"), 1971, p. 118-122. (edited translation, Report No. FTD-HC-23-2389-74, Foreign Technology Division, Wright-Patterson AFB, Ohio. (AD/A-000 125/5ST).

West, J. M. and B. C. Slocombe, "Dynamics Consolidation as an Alternative Foundation," Ground Engineering, Vol. 6, No. 6, November, 1973.

Other Methods

Dearstyne, S. C., and G. J. Newman, "Subgrade Stabilization under an Existing Runway," Journal of the Aero-Space Transport Division, ASCE, Proceedings Paper 3473, April 1963, 8 pp.

"Wicks go down; site dries up," Engineering News Record, Vol 206, No. 2, January 8, 1981, p. 26.

Johnson, S. J., "Foundation Precompression with Vertical Sand Drains," Journal of the Soil Mechanics and Foundation Division, ASCE, Vol. 96, No. SM1, Proceeding Paper 7019, January, 1970, pp. 145-175.

Johnson, S. J., "Precompression for Improving Foundation Soils," Journal of the Soil Mechanics and Foundation Division, ASCE, Vol 96, No. SM1, Proceeding Paper 7020, January, 1970, pp. 111-144.

Mancini, F. P. and C. E. O'Bannon, "Field Stabilization of Chinle Clay by Electro-Osmosis and Base Exchange of Ions," U.S. Department of Transportation, Federal Highway Administration, Washington, D.C., 1973, FWHA/AZ/RD-145, 138 pp.

McKinney, R. L., J. E. Kelly, and C. McDowell, "The Waco Ponding Project," Highway Research Record, No. 560, 1976, pp. 31-43.

Bridge Approch Design and Construction Practices, NCHRP, Synthesis of Highway Practice No. 2, Washington, D.C.: Highway Research Board, 1969, 30 pp.

O'Bannon, C. E., G. R. Morris, and F. P. Mancini, "Electrochemical Hardening of Expansive Clays," Transportation Research Bulletin, No. 593, 1976, p. 46-50.

Peck, R. B., "Relocation of Railroad Embankment, O'Hare Field, Chicago," Proceedings, Vol. 57, Bulletin, No. 528, AREA, February 1956, pp. 649-657.

Sheeran, D. E., and R. N. Yong, Performance of Frost Heave Modifying Chemical Additives at the CN Dorval Test Site, McGill University, Montreal, Prepared for Transport Canada Research and Development Centre, Canadian National Railways, Technical Report FHTD-VIII, TP 1814, October 1978, 88 pp.

Steinberg, M. L., "Ponding an Expansive Clay Cut: Evaluation and Zones of Activity," Transportation Research Record, No. 641, 1977, pp. 61-66.

Weber, W. G., "Performance of Embankments Constructed Over Peat," Journal of the Soil Mechanics and Foundation Division, ASCE, Vol. 95, No. SM1, January 1969, pp. 53-76.

D.3 LAYER INSERTS

Filter Fabric

Benson, G. R., "Interim Guide Specifications for Filter Fabrics," Proceedings, Vol. 80, Bulletin, No. 670, AREA, Nov.-Dec., 1978, pp. 205-208.

Newby, J. E., "In Depth View of Geotextile Subgrade Stabilization," Railway Track and Structures, Vol. 76, No. 4, June 1980, pp. 22-37.

"C&NW Attacks the Tough Spots," Progressive Railroading, October, 1976.

"Engineering Fabrics...Used and Researched by Southern Pacific," Progressive Railroading, Vol. 22, No. 3, March 1979, pp. 52-56.

"Answer Found in Stabilization Fabric," Railway Track and Structures, January, 1978.

"Pumping Track on FEC: Answer found in Stabilization Fabric," Railway Track and Structures, January, 1978, pp. 24-25.

"Stabilization Fabrics: Where They Are in Railroading Today," Railway Track and Structures, June 1978, pp. 42-43.

"Testing of Subgrade Stabilization Moves Ahead," Railway Track and Structures, Vol. 72, No. 7, July 1976, pp. 20--21.

"Testing of Subgrade Stabilization Fabrics Moves Ahead-2," Railway Track and Structures, Vol. 72, No. 9, September 1976, pp. 32-34.

"Testing of Subgrade Stabilization Fabrics Moves Ahead-3," Railway Track and Structures, Vol. 72, No. 10, October 1976, pp. 22-23.

"Testing of Subgrade Stabilization Fabrics Moves Ahead-4," Railway Track and Structures, Vol. 72, No. 12, December 1976, pp. 18-19.

Other Methods

Barkey, B. A., "Use of Asphalt Membranes to Reduce Expansion in Certain Types of Expansive Soils," Proceedings American Association of State Highway Officials, 53rd Annual Meeting, Salt Lake City, Utah, October, 1967.

Bell, J. R., and E. J. Yoder, "Plastic Moisture Barriers for Highway Subgrade Protection," Proceedings, Vol. 36, Highway Research Board, 1957, pp. 713-735.

Burns, C. D. and W. N. Brabstone, Membrane Envelope Technique for Waterproofing Soil Base Courses for Airstrips; Base Support, Vicksburg: Waterways Experiment Station, Miscellaneous Paper S-68-13, 1968.

Bituminous Emulsions for Highway Pavements, NCHRP, Report No. 30, Washington, D.C.: Transportation Research Board, 1975, 76 pp.

Insulation

Hartmark, H., "Frost Protection of Railway Lines," Engineering Geology, 1979, Vol. 13, pp. 505-517.

Horton, J. A., M. M. Bowers, and C. W. Lovell, "Indiana's Thermally Insulated Test Road," Highway Research Record, No. 429, 1973, pp. 47-54.

Keiner, E. P., "Insulated and Improved Subgrades for Eliminating Frost Heave," Civil Engineering, ASCE, February 1969, pp. 38-39.

Ministry of Transport of the USSR, Tekhnicheskie ukazaniya po ozdorovleniyu osnovnoi ploshchadki semlyanogo polotna puchinnykh uchastkov (Technical Instructions on Treatment of the Roadbed Subgrade in Heaving Sectors), Publishing House "Transport", Moscow, 1968, 126 pp., Translation Bureau, Canada, Bureau No. 769613.

Peckover, F. L., "Roadway Stabilization and Drainage," draft submitted for Track Cyclopedia, March, 1978.

Penner, E., "Insulated road study," Transportation Research Record, No. 612, 1976, pp. 80-83.

Skaven-Haug, S., "Protection against frost heaving on Norwegian railways," Geotechnique, Vol. IX, No. 3, September 1959, p. 87.

Spang, J., "Planumschutzschichten unter Eisenbahngleisen," ("Layers protecting the track formation"), Archiv fur Eisenbahntechnik, November 1965, pp. 181-211, (Translation No. 667 of British Railway Research Department).

D.4 COMPACTION AND ADMIXTURES

American Railway Engineering Association, "Lime-Soil Stabilization Investigation for Railroad Construction," Proceedings, Vol. 69, Bulletin, No. 612, AREA, February, 1968, pp. 516-527.

Fundamentals of Soil Compaction, Highway Research Bulletin, No. 272, Washington D.C.: Highway Research Board, 1960, pp. 4-19.

Stabilized Soils: Mix Design and Properties - 6 Reports, Highway Research Record, No. 198, Highway Research Board, 1967, 70 pp.

Ino, T., "Reinforced Subgrade," _Japanese Railway Engineering_, Vol. 17, No. 1, 1977, pp. 22-23.

Ireland, H. O., "Soil Stabilization Lime Used for Soil Stabilization," _Proceedings_, Vol. 71, _Bulletin_, No. 628, AREA, June 1970, pp. 846-855.

Joshi, R. C., D. M. Duncan, and H. M. McMaster, "New and Conventional Engineering Use of Fly-Ash," _Transportation Engineering Journal_, ASCE, Vol. 101, No. TE4, 1975, pp. 791-806.

Lime Stabilization Construction Manual, No. 326, 5th revised edition, Washington D.C.,: National Lime Association, 1972, 46 pp.

"Lime Does Job For MoPac," _Progressive Railroading_, September 1978.

State of the Art: Lime Stabilization, Reactions, Properties, Design, Construction, Transportation Research Circular, No. 180, Washington, D.C., Transportation Research Board, September, 1976, 31 pp.

D.5 EMBANKMENT REDESIGN

Retaining Structures

"Roadbed Stabilization," _Proceedings_ of the Fifty-Third Annual Convention of the AREA, Vol. 55, 1954, pp. 637-638.

de Beer, E. E. and M. Wallays, "Stabilization of a Slope in Schists by Means of Bored Piles Reinforced with Steel Beams," _Proceedings_, 2nd Congress of the International Society Rock Mechanics, Belgrade, Theme 7, No. 13, 1970, 9 pp.

Farris, J. B., "Roadbed Stabilization - Various Methods," _Proceedings_, Vol. 76, _Bulletin_, No. 651, AREA, 1975, pp. 419-431.

Lizzi, F., "Reticulated Root Piles - To Correct Landslides," _ASCE_ Convention and Exposition, Chicago, Illinois, Preprint 3370, October, 1978, 25 pp.

Munoz, A. and R. M. Mattox, "Vibroreplacement and Reinforced Earth Unite to Strengthen a Weak Foundation," Civil Engineering, ASCE, Vol. 47, No. 5, 1977, pp. 58-62.

Walkinshaw, J. L., "Reinforced Earth Construction," U.S. Department of Transportation, Federal Highway Administration, Washington, D.C., April 1975, FHWA-DP-18.

Widen, Lower, Lightweight Fill

Farris, J. B., "Roadbed Stabilization - Various Methods," Proceedings, Vol. 76, Bulletin, No. 651, AREA, 1975, pp. 419-431.

Ireland, H. O., "Soil Problems in Railway Transportation Engineering," Proceedings, Vol. 67, Bulletin, No. 594, AREA, September-October 1965, pp. 7-19.

Monahan, E. J., "Weight-Credit Foundation Construction Using Foam Plastic as Fill," Proceedings, International Symposium on New Horizons in Construction Materials Bethlehem, Pa.,: Envo Publishing Co., Vol. 1, 1976, pp. 199-210.

Thurber, R. C., "Some Methods of Construction in Peat Bogs and Swamps in British Columbia," Proceedings, 10th Muskeg Research Conference, National Research Council, Canada, May, 1964, pp. 59-69.

Vertical Reinforcement

"Machinery and Equipment for Stabilizing Roadbed," Proceedings of the Forty-Seventh Annual Convention of the AREA, Vol. 49, 1948, pp. 167-171.

Englehardt, K. et al., "Vibro-Replacement - A Method to Strengthen Cohesive Soils In-Situ," National Structural Engineering Meeting, ASCE, Cincinnati, Ohio, Preprint 2281, April 1974, 30 pp.

Watzlaw, W., "Pfahlgruendung eines Eisenbahndammes," ("A Railway Embankment on a Pile Foundation"), Eisenbahntechnische Rundschau, Vol. 24, No. 11, November 1975, pp. 409-416.

Excavate Replace

Casagrande, A., "An Unsolved Problem of Embankment Stability on Soft Ground," Proceedings, 1st Pan American Conference on Soil Mechanics, Mexico City, Vol. 2, pp. 721-746.

APPENDIX E

REPORT OF NEW TECHNOLOGY

The findings of this study are based on a review of published literature, discussions with practicing engineers, and our own experience. As such, no novel technologies have been developed.

The purpose of this study is to determine the state-of-the-art of earth materials practices as they may be applied to railroad substructure engineering and subgrade stabilization. The technology of railroad engineering, pavement engineering, geology, and soil mechanics were drawn on to develop recommended materials practices for dealing with track substructures. Thus, it is proposed to apply existing technologies to new uses.

Recommendations for applying standard soil testing procedures to classifying and characterizing subgrade soils is contained in Table 2-15. Recommendations for classifying and characterizing ballast materials are contained in Table 3-2 and Figures 3-2 and 3-3. Recommendations for classifying and identifying subballast are shown in Figure 4-1. A basis for selection of subballast is contained in Section 4.3 and Figures 4-2 and 4-3.

A broad selection of subgrade stabilization methods was reviewed in Section 5. These methods have been evaluated with respect to their application to railroad substructure engineering, in particular, non-disruptive improvements to track subgrades.

BIBLIOGRAPHY

Note: References related to stabilization are given in Appendix D. Citations included in the Appendix are not repeated in this Bibliography.

Ahlf, R.E., "Matching Maintenance of Way Practice to Required Use of Track -- Part IV: the Costs," Railroad Track and Structures, Vol. 75, No. 11, January, 1980, pp. 21-23.

American Railway Engineering Association, Manual for Railway Engineering, Chapter 1, Parts 1 and 2, 1978, 78pp.

American Society for Testing and Materials, Annual Book of ASTM Standards, Part 14: Concrete and Mineral Aggregates, 706pp.; Part 19: Soil and Rock; Building Stone, Peat, 486 pp., Philadelphia, 1979.

Atterberg, A., "Die Plastizitat der Tone." Int. Mitt. fur Bodenhunde, Vol. 1, 1911, pp. 10-43.

Aufmuth, R.E., "Site engineering indexing of rock," ASTM Special Technical Publication Number STP 554, Philadelphia, 1974, pp. 81-99.

Azzouz, Amr S. et al., "Regression Analysis of Soil Compressibility," Soils and Foundations, Vol. 16, No. 2, June 1976, pp. 19-29.

Bell, F.G., ed., Methods of Treatment of Unstable Ground, London: Butterworths & Company, Limited, 1975, 215pp.

Berg, R.L., "Design of Civil Airfield Pavements for Seasonal Frost and Permafrost Conditions," U.S. Army Engineer Waterways Experiment Station, Soils and Pavements Laboratory, Vicksburg, Mississippi, Prepared for U.S.D.O.T., Federal Aviation Administration, Systems Research & Development Service, Washington, D.C., Final Report No. FAA-RD-74-30, October 1974, 92 pp. (AD-A006284/4GL).

Bishop, A.W. and D.J. Henkel, The Measurement of Soil Properties in the Triaxial Test, St. Martins Press, New York, 1962, 227 pp.

Bjerrum, L., "Embankments on Soft Ground," Proceedings of the Specialty Conference on Performance of Earth and Earth- Supported Structures, Purdue University, Vol. 2, 1972, pp. 1-54.

Bjerrum, L. and A. Landva, "Direct Simple Shear Tests on a Norwegian Quick Clay," Geotechnique, Vol. 16, No. 1, 1966, pp. 1-20.

Bosserman, B., "Ballast Gradation Measurement Procedure," Specification for FAST Ballast Experiment, Pueblo, CO: U.S.D.O.T., Transportation Test Center, 1978, 5pp.

British Standards Institute Standard Tests, Vol. 812: Part 1: 1975 Sampling Size, Shape and Classification, August 1976, 24 pp. (ISBN: 0 580 08744 1); Vol. 812: Part 2: 1975-- Physical Properties, August 1976, 20 pp., (ISBN: 0 580 08754 9).

Brock, E. and J.A. Franklin, "The Point-Load Strength Test." International Journal of Rock Mechanics and Mining Science, Vol. 9, 1972, pp. 669-697.

Burmister, D.M., "Identification and Classification of Soils: An Appraisal and Statement of Principles." Proceedings, Symposium on Identification Classification of Soils, ASTM Special Technical Publication STP 113, 1950, pp. 3-24.

Casagrande, A., "The Determination of the Preconsolidation Load and Its Practical Significance," Proceedings of the First International Conference on Soil Mechanics and Foundation Engineering, Cambridge, Mass., 1936, pp. 1-60.

Casagrande, A., "Classification and Identification of Soils," Transactions, ASCE, 1948, pp. 901-992.

Chapman, D.R., et al., "A Comparative Study of Shale Classification Tests and Systems," Association of Engineering Geologists Bulletin, Vol. 13, No. 4, 1976, pp. 247-266.

Chan, L-S, "An Investigation of Stress-Strain and Strength Characteristics of Cohesionless Soils by Triaxial Compression Tests," Proceedings of the Second International Conference on Soil Mechanics and Foundation Engineering, Rotterdam, 1948, p. 35.

ConRail (Consolidated Rail Corporation), "Specifications for Stone Ballast," Philadelphia, 1974, 3pp.

Cornforth, D.H., "Prediction of Drained Strength of Sands from Relative Density Measurements," ASTM Special Technical Publication STP 523, Philadelphia, 1973, pp. 281-303.

Cornforth, D.H., "Some Experiments on the Influence of Strain Conditions on the Strength of Sand," Geotechnique, Vol. 116, 1964, p. 193.

Coulomb, C.A., "Essai sur une application des règles de maximis et minimis à quelques problèmes de statique relatifs à l'architecture," Mem. Div. Savants, Acad. Sci., Vol. 7, Paris, 1776.

Dalton, C.J., Field Durability Tests on Ballast Samples as a Guide to the Significance of the Specification Requirements, Canadian National Railways Technical Research Center, St. Laurent, Quebec, January 1973, 40 pp.

Dalton, C.J., Strength Tests on Crushed Rock Ballast, Canadian National Rail Research Center, Montreal, Quebec, 1977, 18 pp.

D'Appolonia, D., H. Poulos, and C. Ladd, "Initial Settlement of Structures on Clay," Journal of the Soil Mechanics and Foundations Division, ASCE, Vol. 97, No. SM10, Proc. Paper 8438, October 1971, pp. 1359-1377.

de Mello, V.F.B., "The Standard Penetration Test," Proceedings of the Fourth Pan American Conference on Soil Mechanics and Foundation Engineering, ASCE, Puerto Rico, Vol. 1, 1971, pp. 1-86.

Dowding, C.H., ed., "Site Characterization and Exploration," ASCE, 1979, New York, 395 pp.

"Effects of Different Methods of Stockpiling Aggregates," Interim Report, NCHRP Report No. 5, Highway Research Board, Washington, D.C., 1964, 48 pp.

Epps, J.A., et al., "Soil Stabilization: A Mission Oriented Approach," Highway Research Record, No. 351, 1971, pp. 1-20.

Epps, J.A., W.A. Dunlap, and B.M. Gallaway, "Basis for the Development of a Soil Stabilization Index System," Air Force Systems Command, Air Force Weapons Laboratory, Kirkland, New Mexico, Report No. AFWL-TR-70-176, December, 1971, 271 pp. (AD-735295).

Gaskin, P.N., and G.P. Raymond, "Contribution to Selection of Railroad Ballast," Transportation Engineering Journal, ASCE, Vol. 102, No. TE2, Proc. Paper 12134, May 1976, pp. 377-394.

Goldbeck, T., "Mineral aggregates for railroad ballast." ASTM Special Technical Publication STP 83, 1948, pp. 197-204.

Grim, R.E., Applied Clay Minerology, McGraw-Hill Co., New York, 1962, 422 pp.

Gromko, G.J., "Review of Expansive Soils," Journal of the Geotechnical Engineering Division, ASCE, Vol. 100, No. GT6, 1974, pp. 667-687.

Hardin, B.O., and F.E. Richart, Jr., "Elastic Wave Velocities in Granular Soils," Journal of the Soil Mechanics and Foundations Division, ASCE, Vol. 89, No. SM1, Proc. Paper 3407, February 1963, pp. 33-35.

Hay, W.W., R. Baugher, and A.J. Reinschmidt, "A Study of Railroad Ballast Economics," U.S. Department of Transportation, Federal Railroad Administration, Washington, D.C., September 1977, FRA/ORD-77/65, 91 pp. (PB 275 102AS)*

---

*Numbers in parenthesis at the end of an entry indicate the NTIS order number for the publication.

Hazen, A., "Discussion of 'Dams on Sand Foundations' by A.C. Koenig," Transactions, ASCE, Vol. 73, 1911, p. 199.

Highway Research Board, Effects of Different Methods of Stockpiling and Handling Aggregates, NCHRP No. 46, Highway Research Board, Washington, D.C., 1967, 102 pp.

Holtz, W. and Gibbs, H., "Engineering Properties of Expansive Clays," Transactions, ASCE, Vol. 121, 1956, pp. 641-677.

Hough, B.K., Basic Soils Engineering, Ronald Press, New York, 1957.

Huang, E.Y., "A Test for Evaluating the Geometric Characteristics of Coarse Aggregate Particles," Proceedings, ASTM, Vol. 62, 1962, pp. 1223-42.

Hudson, S.B. and H.F. Waller, Evaluation of Construction Control Procedures: Aggregate Gradation Variations and Effects, NCHRP No. 69, Highway Research Board, Washington, D.C., 1969, 63 pp.

Hughes, B.P. and B. Bahramian, "Laboratory Test for Determining the Angularity of Aggregate." Magazine of Concrete Research, Vol. 6, No. 18, September 1966, pp. 147-152.

Hvorslev, M.J., Subsurface Exploration and Sampling of Soils for Civil Engineering Purposes, Report of the Committee on Sampling and Testing, Soil Mechanics and Foundations Division, ASCE, 1948, 521 pp.

Hvorslev, M.J., Time Lag and Soil Permeability in Groundwater Observations, Waterways Experiment Station, Vicksburg, Bulletin No. 36, 1951.

Ingles, O.G. and J. B. Metcalf, Soil Stabilization: Priniciples and Practice, John Wiley & Sons, New York, 1973, 293pp.

International Society for Rock Mechanics, Commission on Standardization of Laboratory and Field Tests, Committee on Laboratory Tests, Suggested Methods for Determining Water Content, Porosity, Density, Absorption and Related Properties and Swelling and Slake-Durability Index Properties. Document No. 2, November 1972, 36 pp.

International Society for Rock Mechanics, Commission on Standardization of Laboratory and Field Tests, Committee on Field Tests, Suggested Methods for Determining Shear Strength. Document No. 1, February 1974, 35 pp.

International Union of Railways, Classification of Ballast Prescriptions, Interim Report D71/RP 7/E, June 1967, 62 pp.

International Union of Railways, Stresses in the Rails, the Ballast, and the Formation Resulting from Traffic Loads: Summary of the Results Given in Reports Nos. 1-2, Interim Report D71/RP 13/E, October 1970, 31 pp.

Jenkins, H.H., "Track Maintenance for High Speed Trains," Proceedings, Vol. 77, Bulletin, No. 658, AREA, 1976, pp. 499-521.

Kaplar, C.W., Freezing Test For Evaluating Frost Susceptibility of Various Soils, Cold Region Research Engineering Laboratory Technical Report No. 250, 1974, 40 pp. (AD 781511.)

Kassiff, G., M. Livneh, and G. Wiseman, Pavements on Expansive Clays, Jerusalem Academic Press, Jerusalem, 1969, 218 pp.

Knutson, R.M., et al., "Materials Evaluation Study," Ballast and Foundation Materials Research Study, U.S. Department of Transportation, Federal Railraod Administration, Washington, D.C., January 1977, FRA/ORD-77/02, 324 pp. (PB264-215).

Ladd, C.C., "Measurement of In-Situ Shear Strength," Discussion of a state-of-the-art paper by J.H. Schmertmann, Proceedings of the ASCE Specialty Conference on In-Situ Measurement of Soil Properties, Vol. 2, 1975, pp. 152-160.

Ladd, C.C., et al., "Stress-Deformation and Strength Characteristics," Proceedings of the Ninth International Conference on Soil Mechanics and Foundation Engineering, Vol. 2, Japanese Society of Soil Mechanics and Foundation Engineering, Tokyo, 1977, pp. 421-494.

Ladd, C.C. and L. Edgers, Consolidated-Undrained Direct-Simple Shear Tests on Saturated Clays, Department of Civil Engineering, M.I.T., Research Report R72-82, 1972, 354 pp.

Ladd, C.C. and R. Foote. "New Design Procedure for Stability of Soft Clays," Journal of the Geotechnical Engineering Division, ASCE, Vol. 100, No. GT-7, 1974, pp. 763-786.

Ladd, C.C. and T.W. Lambe, "The Strength of Undisturbed Clay Determined from Undrained Tests," ASTM-NRC Symposium, ASTM Special Technical Publication STP 361, 1963, pp. 342-371.

Lambe, T.W., Soil Testing for Engineers, John Wiley & Sons, New York, 1951, 165 pp.

Lambe, T.W. and R.V. Whitman, Soil Mechanics, John Wiley & Sons, New York, 1969, 553 pp.

Lee, K.L. and H.B. Seed, "Dynamic Strength of Anisotropically Consolidated Sand," Journal of the Soil Mechanics and Foundations Division, ASCE, Vol. 93, No. SM-5, part 1, 1967, pp. 169-190.

Lees, G., "The Measurement of Particle Elongation and Flakiness: A Critical Discussion of British Standard and Other Methods," Magazine of Concrete Research, Vol. 16, No. 49, 1964, pp. 225-230.

Linell, K.A., F.B. Hennion, and E.F. Lobacz, "Corp of Engineers' Pavement Design in Areas of Seasonal Frost," Highway Research Record, No. 33, 1963, pp. 76-136.

Liu, T.K., "A Review of Engineering Soil Classification Systems," Highway Research Record, No. 156, pp. 1-22.

Loudon, A.G., "The Computation of Permeability from Simple Soil Tests," Geotechnique, Vol. 3, 1953, pp. 165-183.

Mitchell, J. and W. Gardner, "In-Situ Measurement of Volume Change Characteristics," Proceedings of the ASCE Specialty Conference on In-Situ Measurement of Soil Properties, Vol. 2, 1975, pp. 279-345.

Mitchell, J.K., "In-place Treatment of Foundation Soils," Journal of the Soil Mechanics and Foundation Engineering Division, ASCE, Vol. 96, No. SM 1, January, 1970, pp. 73-110.

Nascimento, V. and S. Simoes, "Relation between CBR and Modulus of Strength," Proceeding of the Fourth International Conference on Soil Mechanics and Foundation Engineering, Vol. 2, 1957, pp. 166-168.

Nichols, F.P., Jr., "Review of History of Old Cementing Value Test for Railroad Ballast," Washington, D.C., National Crushed Stone Association, Private Communique, 1975.

Nichols, F.P., Jr., "Toward Better Evaluation of Railroad Ballast Aggregate," Washington, D.C., National Crushed Stone Association, Research Report, April 1977, 4pp.

Nishiki, T., "How to Choose Proper Surface Soils for Roadbeds," Permanent Way (Japan), No. 15, June, 1962, pp. 1-8.

O'Neill, H.M., "Direct Shear Test for Effective Strength Parameters," Journal of the Soil Mechanics and Foundations Division, ASCE, Vol. 88, No. SM-4, 1962, pp. 109-137.

Paterson, A., "Matching the Track to the Load," Railway Gazette International, Vol. 128, No. 2, February 1972, pp. 53-56.

Peck, R.B., "Progress Report of the Joint Investigation of Methods of Roadbed Stabilization," Proceedings, AREA, Vol. 47, No. 458, February, 1946, pp.682-689.

Peck, R.B. et al., Foundation Engineering, Second Edition, John Wiley & Sons, New York, 1974, 514 pp.

Peckover, F.L., Railway Ballast Materials, Canadian National Railways, Montreal, June 1973, 97 pp.

Peckover, F.L., "European Ballast Practice as Discussed by Dr. Riessberger of Austria," Interdepartmental Correspondence, 4 April 1973, Canadian National Railroad, Montreal, Engineering Department.

Peckover, F.L., "Frost Heaving of Track - Causes and Cures," AREA Bulletin, No. 666, 1976, pp. 143-173.

Pettijohn, F.J., Sedimentary Rocks, 2nd ed., Chapter 2, Harper Bros., New York, 1957, 718 pp.

Raymond, G.P., Stresses and Deformation in Railway Track, CIGGT Report No. 77-15, Queens University, Kingston, Ontario, November 1977, 168 pp.

Raymond, G.P., "Railroad Ballast Prescription: State-of-the-Art," Journal of the Geotechnical Engineering Division, ASCE, Vol. 105, No. GT2, Proceedings Paper 14397, February 1979, pp. 305-322.

Raymond, G.P., "Ballast Properties That Affect Ballast Performance," American Railway Engineering Association-Bulletin, Vol. 80, No. 673, June-July 1979, pp. 428-447.

Raymond, G.P., P.N. Gaskin, and O. Svec, "Selection and Performance of Railroad Ballast," in Railroad Track Mechanics and Technology, A.D. Kerr, ed., Pergamon Press, Princeton University, October 1975, pp. 369-386.

Raymond, G.P., et al., A Study of Stresses and Deformations Under Dynamic and Static Load Systems in Track Structure and Support, CIGGT Report No. 75-10, Queen's University, Kingston, Ontario, September 1975, 53 pp.

Raymond G.P., et al., "Characteristics of Acceptable Railroad Ballast," Paper Presented to Session Four, Transportation Research Board, January 24, 1977, Sponsored by Mineral Aggregates Committee, Washington, D.C.

Raymond, G.P., R.W. Lake, and C.J. Boon, Stress and Deformations in Railroad Track, CIGGT Final Report No. 76-11, November 1976, 171 pp.

Rhoades, R. and R.C. Mielenz, "Petrographic and Mineralogic Characteristics of Aggregates," ASTM Special Technical Publication, STP 83, 1948, pp. 20-48.

Robnett, Q.L., et al., "Technical Data Bases Report," U.S. Department of Transportation, Federal Railroad Administration, Washington, D.C., July 1975, FRA/ORD-76/138, 179 pp.

Roscoe, K.H., "An Apparatus for the Application of Simple Shears to Soil Samples," Proceedings of the Third International Conference on Soil Mechanics and Foundation Engineering, Switzerland, Vol. 1, 1953, pp. 186-191.

Rowe, P.W., "The Stress-Dilatency Relation for Static Equilibrium of an Assembly of Particles in Contact," Proceedings of the Royal Society, A269, 1962, pp. 500-527.

Sanglerat, G., The Penetrometer and Soil Exploration. Elsevier Publishing Co., New York, 1972, 432 pp.

Schmertmann, J.H., Guidelines for Cone Penetration Test Performance and Design. Prepared for the U.S.D.O.T. Federal Highway Administration, 1977, FHWA-TS-78-209, 145 pp.

Schmertmann, J.H., "The Undisturbed Consolidation of Clay," Transactions, ASCE, 1955, p. 1201.

Schmidt, B., et al., "Subsurface Exploration Methods for Soft-Ground Rapid Transit Tunnels," Vol. 1, U.S. Department of Transportation, Urban Mass Transportation Administration, Washington, D.C., 1976, UMTA-MA-06-0025-76-1, 202 pp.

Schramm, D., Permanent Way Technique and Permanent Way Economy. Otto Elsner Verlaggesellschaft, Darmstadt, 1961, pp. 97-116, 203-218.

Selig, E.T., T-S. Yoo, and C.M. Panuccio, Field Methods for Ballast Physical State Measurements, Mechanics of Ballast Compaction Study, Phase I, prepared for U.S. Dept. of Transportation, Federal Railroad Administration, Report No. FRA/ORD 81/16.2, March 1982, 155 pp.

Selig, E.T., T-S Yoo, and C.M. Panuccio, Technical Review of Ballast Compaction and Related Topics, Mechanics of Ballast Compaction Study - Phase I, Vol. 1, Unpublished Interim Report prepared for U.S.D.O.T. Transportation Systems Center, Report No. FRA/ORD 81/16.1, March 1982, 287 pp.

Selig, E.T. et al., "A Theory for Track Maintenance Life Prediction," U.S. Department of Transportation, Research and Special Programs Administration, Washington, D.C., 1979, DOT-RSPA-DPB-50-79-22, 183 pp.

Sluz, A., "Ballast Sampling Procedure for Ballast Unloading and Stockpiling," Specification for FAST Track, U.S.D.O.T, Transportation Test Center, Pueblo, CO., 1979, 3pp.

Soil Improvement, Report of the Committee on Placement and Improvement of Soils, ASCE, 1978, 182pp.

Spang, J., "Planumschutzschichten unter Eisenbahngleisen" (Layers protecting the track formation). Archiv fur Eisenbahntechnik, November 1965, pp. 181-211. (Translation No. 667 of the British Railway Research Department)

Sridharan, A. and S.N. Rao, "The Relationship Between Undrained Strength and Plasticity Index," Geotechnical Engineering, Vol. 4, 1973, pp. 41-53.

Sweet, H.S., "Physical and Chemical Tests of Mineral Aggregates and Their Significance," ASTM Special Technical Publication STP 83, Philadelphia, 1948, pp. 49-73.

Talbot, A.N. and F.E. Richart, The Strength of Concrete, Its Relation to Cement, Aggregate and Waters, Bulletin 137, Engineering Experiment Station, University of Illinois, Urbana, October 1923, 118 pp.

Terzaghi, K., "Past and Future of Applied Soil Mechanics," Contributions to Soil Mechanics, Boston Society of Civil Engineers, 1965, pp. 400-444.

Terzaghi, K. and R.B. Peck, Soil Mechanics in Engineering Practice, John Wiley & Sons, New York, 1967, 727 pp.

Thompson, M.R., W.W. Hay and S.D. Tayabji, "Ballast and Foundation Materials Research Study," U.S. Department of Transportation, Federal Railroad Administration, Washington, D.C., June 1978, FRA/ORD-78/10, 84 pp. (PB 282348).

Titov, V.I., ed., "The Study of Heave Control Measures for Railways," The All-Union Railway Transport Research Institute, Moscow, USSR, 1970, 212pp.

Toms, A.H. and D.L. Barlett, Proceedings, The Institution of Civil Engineers, Vol. 21, April, 1962, pp. 705-732.

U.S. Department of the Interior, Earth Manual, A Water Resources Technical Publication, Second Edition, 1974, 810 pp.

U.S. Department of the Navy, Civil Engineering Design Manual, NAVFAC DM-5, 1974, 387 pp.

U.S. Department of the Navy, Soil Mechanics, Foundations, and Earth Structures Design Manual, NAVFAC DM-7, 1971, 314 pp.

U.S. Federal Aviation Administration, "Airport Paving," Advisory Circular, 150/5320-6, 1974, 29 pp.

Voight, B., "Correlation Between Atterberg Plasticity Limits and Residual Shear Strength of Natural Soils," Geotechnique, Vol. 23, No. 2, 1973, pp. 265-267.

West, T.R. et al., "Tests for Evaluating Degradation of Base Course Aggregates," NCHRP No. 98, Washington, D.C., Highway Research Board, 1970, 98 pp.

Wissa, A.E.Z., R.T. Martin and J.E. Garlanger, "The Piezometer Probe," Proceedings of the Conference on In-Situ Measurement of Soil Properties, ASCE, Raleigh, N.C., 1975, pp. 536-545.

Woods, R.D., "Measurement of Dynamic Soil Properties," Proceedings of the ASCE Specialty Conference on Earthquake Engineering and Soil Dynamics, Pasadena, California, Vol. 1, 1978, pp. 91-178.

Woolfe, D.O., "Toughness, Hardness, Abrasion, Strength and Elastic Properties," ASTM Special Technical Publication STP 169A, 1966, pp. 462-475.

Yoder, E.J., Review of Soil Classification Systems Applicable to Airport Design, Prepared for the U.S.D.O.T. Federal Aviation Agency, System Research and Development Service, May 1974, 122 pp. (AD-783 190)

Yoder, E.J. and M.W. Witczak, Principles of Pavement Design, 2nd edition, John Wiley & Sons, New York, 1975, 569 pp.

Yudbhir, "Field Compressibility of Soft Sensitive Normally Consolidated Clays," Geotechnical Engineering, Vol. 4, No. 1, 1973, pp. 31-40.