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Ground Modification Methods Reference Manual – Volume I






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NOTICE

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16. Abstract

This FHWA Geotechnical Engineering Circular No. 13 provides guidance on Ground Modification Methods, and also serves as the reference manual for FHWA NHI courses No. 132034, 132034A, and 132034B on Ground Modification Methods. The purpose of this manual is to introduce available ground modification methods and applications to design generalists, design specialists, construction engineers, and specification and contracting specialists involved with projects having problematic site conditions.

An introductory chapter provides a description, history, functions, and categories of ground modification. A description of the web-based *GeoTechTools* (http://www.geotechtools.org) technology selection guidance system and geotechnology catalog is also provided in the first chapter. The introductory chapter is followed by stand-alone technical category chapters. Each category chapter includes a broad introduction to the technical category including typical applications, a listing of common technologies used in the U.S., and summaries for specific technologies in the category. Each technology summary includes: description; advantages and limitations; applicability; complementary technologies; construction methods and materials; photographs; design guidance; quality assurance methods; costs; specifications; and reference list. Each technical category and the technology summaries therein reflect current practice in design, construction, contracting methods, and quality procedures. This publication was prepared with the practicing transportation specialist in mind and with the benefit of extensive industry review.

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SI* (MODERN METRIC) CONVERSION FACTORS

APPROXIMATE CONVERSIONS TO SI UNITS

Symbol	When You Know	Multiply By	To Find	Symbol
		LENGTH		
in	inches	25.4	millimeters	mm
ft	feet	0.305	meters	m
yd	yards	0.914	meters	m
mi	miles	1.61	kilometers	km
		AREA		
in^2	square inches	645.2	square millimeters	mm^2
ft^2	square feet	0.093	square meters	m^2
yd^2	square yard	0.836	square meters	m^2
ac	acres	0.405	hectares	ha
mi^2	square miles	2.59	square kilometers	km^2
		VOLUME		
fl oz	fluid ounces	29.57	milliliters	mL
gal	gallons	3.785	liters	L
ft^3	cubic feet	0.028	cubic meters	m^3
yd^3	cubic yards	0.765	cubic meters	m^3
		NOTE: volumes greater than 1000 L shall be shown in m^3		
		MASS		
oz	ounces	28.35	grams	g
lb	pounds	0.454	kilograms	kg
T	short tons (2000 lb)	0.907	megagrams (or "metric ton")	Mg (or "t")
		TEMPERATURE (exact degrees)		
^oF	Fahrenheit	5 (F-32)/9 or (F-32)/1.8	Celsius	^oC
		ILLUMINATION		
fc	foot-candles	10.76	lux	lx
fl	foot-Lamberts	3.426	candela/m^2	cd/m^2
		FORCE and PRESSURE or STRESS		
lbf	poundforce	4.45	newtons	N
lbf/in^2	poundforce per square inch	6.89	kilopascals	kPa

APPROXIMATE CONVERSIONS FROM SI UNITS

Symbol	When You Know	Multiply By	To Find	Symbol
		LENGTH		
mm	millimeters	0.039	inches	in
m	meters	3.28	feet	ft
m	meters	1.09	yards	yd
km	kilometers	0.621	miles	mi
		AREA		
mm^2	square millimeters	0.0016	square inches	in^2
m^2	square meters	10.764	square feet	ft^2
m^2	square meters	1.195	square yards	yd^2
ha	hectares	2.47	acres	ac
km^2	square kilometers	0.386	square miles	mi^2
		VOLUME		
mL	milliliters	0.034	fluid ounces	fl oz
L	liters	0.264	gallons	gal
m^3	cubic meters	35.314	cubic feet	ft^3
m^3	cubic meters	1.307	cubic yards	yd^3
		MASS		
g	grams	0.035	ounces	oz
kg	kilograms	2.202	pounds	lb
Mg (or "t")	megagrams (or "metric ton")	1.103	short tons (2000 lb)	T
		TEMPERATURE (exact degrees)		
^oC	Celsius	1.8C+32	Fahrenheit	^oF
		ILLUMINATION		
lx	lux	0.0929	foot-candles	fc
cd/m^2	candela/m^2	0.2919	foot-Lamberts	fl
		FORCE and PRESSURE or STRESS		
N	newtons	0.225	poundforce	lbf
kPa	kilopascals	0.145	poundforce per square inch	lbf/in^2

* SI is the symbol for the International System of Units. Appropriate rounding should be made to comply with Section 4 of ASTM E380. (Revised March 2003)

PREFACE

One of the major tasks within geotechnical engineering is to design, implement and evaluate ground modification schemes for infrastructure projects. During the last forty years significant new technologies and methods have been developed and implemented to assist the geotechnical specialist in providing cost-effective solutions for construction on marginal or difficult sites.

The impetus for ground modification has been both the increasing need to use marginal sites for new construction purposes and to mitigate risk of failure or of poor performance. During the past several decades, ground modification has come of age and reached a high level of acceptance in the geotechnical community. Its use is now routinely considered on most projects where poor or unstable soils are encountered. From the geotechnical engineer's point of view, ground modification means the modification of one or more of the relevant design engineering properties (e.g., increase in soil shear strength, reduction of soil compressibility, and reduction of soil permeability) – or the transfer of load to more competent support layers. From the contractor’s point of view, ground modification may mean a reduction in construction time and/or a reduction in construction costs. Both points of view are valid reasons to consider the use of ground modification techniques and are often mutually inclusive.

Herein, ground modification is defined as the alteration of site foundation conditions or project earth structures to provide better performance under design and/or operational loading conditions. Ground modification objectives can be achieved using a large variety of geotechnical construction methods or technologies that alter and improve poor ground conditions where traditional over-excavation and replacement is not feasible for environmental, technical or economic reasons. Ground modification has one or more of the following primary functions, to:

- increase shear strength and bearing resistance,
- increase density,
- decrease permeability,
- control deformations (settlement, heave, distortions),
- improve drainage,
- accelerate consolidation,
- decrease imposed loads,
- provide lateral stability,

- increase resistance to liquefaction, and/or
- transfer embankment loads to more competent subsurface layers.

The purpose of GEC 13 is to introduce available ground modification methods and applications to design generalists (i.e., project planners, roadway designers, consultant reviewers, etc.), design specialists (i.e., geotechnical, structural, pavement, etc.), construction engineers, specification writers, and contracting specialists involved with projects having problematic site conditions. This publication was prepared with practicing transportation specialists and generalists in mind.

The introductory chapter provides a description, history, functions, and categories of ground modification. Additionally, the role of ground modification in addressing project risks and constraints and risk mitigation, and contracting mechanisms and their impact on selection of ground modification technologies are described. The chapter also includes description of the web-based *GeoTechTools* (http://www.geotechtools.org) technology selection guidance system, and its use for the initial screening process of developing a short-list of technologies applicable to a given project. The *GeoTechTools* geotechnology catalog, of over 50 technologies, and the engineering tools provided for each technology are described. A discussion of final project-specific technology selection that extends beyond the initial screening that can be developed within *GeoTechTools* is included in Chapter 1. Through incorporation of technology and project specific factors, a 12-step process is presented that leads to selection of a preferred, specific technology for a given project.

The introductory chapter is followed by stand-alone technical category chapters. Each category chapter includes a broad introduction to the technical category including typical applications, a listing of common technologies used in the United States, and summaries for specific technologies in the category. Each technology summary includes: description; advantages and limitations; applicability; complementary technologies; construction methods and materials; design guidance; quality assurance methods; costs; specifications; and reference list. Each technical category and the technology summaries therein reflect current practice in design, construction, contracting methods, and quality assurance procedures. Transportation focused case histories are included for select technologies.

This 2016 GEC 13 reference manual on Ground Modification Methods is an update to the 2006 FHWA-NHI-06-019/020 Ground Improvement Methods reference manual. Lead author of the 2006 manual was Victor Elias, PE, and is his last major work. Mr. Elias had a distinguished professional career and provided significant contributions to the design and construction of safe, cost-effective geotechnical works in transportation works. He had been the Principal Investigator for several major research and/or implementation projects focused on durability of soil reinforcement materials, design guidance and specifications for retaining walls foundations and, and ground improvement methods.

Chapters and technology categories contained in this Volume I of the FHWA Ground Modification reference manual set:

Chapters and technology categories contained in the companion Volume II of the FHWA Ground Modification reference manual set:

Chapter 1

INTRODUCTION TO GROUND MODIFICATION TECHNOLOGIES

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1.0 DESCRIPTION AND HISTORY

1.1 Description

When difficult ground conditions are encountered there are a number of alternatives that can be employed to achieve project objectives. These alternatives include: (1) bypassing the poor ground through relocation of the project to a more suitable site or through the use of a deep foundation; (2) removing and replacing the unsuitable soils; (3) designing the planned structure to accommodate the poor/marginal ground; or (4) modifying (improving) the existing soils, either in-place or by removal, treatment and replacement of the existing soils; (ASCE 1978; Mitchell 1981). Through a wide-variety of modern ground improvement and geoconstruction technologies, marginal sites and unsuitable in-situ soils can be improved to meet demanding project requirements, making the latter alternative an economically preferred solution in many cases. In essence, the modern builder has the option to “fix” the poor ground conditions and to make them suitable for the project’s needs (Munfakh and Wyllie 2000). A variety of terms are used to describe this “fixing the ground”: ‘soil improvement’, ‘ground improvement’, ‘ground treatment’, or ‘ground modification’. Charles (2002) notes that the process of altering the ground is *ground treatment*, while the purpose of the process is *ground improvement*, and the result of the process is ***ground modification***. For better or worse the treatment has modified the ground’s support conditions.

Herein, ground modification is defined as the alteration of site foundation conditions or project earth structures to provide better performance under design and/or operational loading conditions (USACE 1999). Ground modification objectives can be achieved using a large variety of geotechnical construction methods or technologies that alter and improve poor ground conditions where replacement is not feasible for environmental, technical or economic reasons. Ground modification has one or more of the following primary functions:

- Increase shear strength and bearing resistance
- Increase density
- Decrease permeability
- Control deformations (settlement, heave, distortions)
- Increase drainage
- Accelerate consolidation
- Decrease imposed loads
- Provide lateral stability
- Increase resistance to liquefaction

- Transfer embankment loads to more competent subsurface layers

There are over four million miles of highways in the United States, including over 164,000 miles on the National Highway System that form the backbone of the public road network (Richard Weingroff, personal communication). The American Society of Civil Engineers 2013 Report Card for America's Infrastructure noted that 32% of America's major roads are in poor or mediocre condition, costing U.S. motorists who are traveling on deficient pavement and bridges $67 billion a year (ASCE 2013). Many miles of these roadways need to be reconstructed, rehabilitated, or upgraded in areas of difficult ground conditions. These efforts increasingly must be done under severe time constraints while minimizing disruption to existing traffic, and with the goal of producing long-lived facilities. The selected course of action must often avoid destruction of or harmful effects to existing, adjacent pavement systems or structural facilities such as bridges, retaining walls, and embankments that still have remaining useful life. The selection of an appropriate geoconstruction technology to use in a transportation project is a complex undertaking that depends upon integration of available knowledge and a number of problem-specific and site-specific constraints and requirements.

1.2 Historical Overview

An early report on soil improvement was that of the ASCE Committee on Placement and Improvement of Soils in 1978 in which it was noted:

> Soil, nature's most abundant construction material, has been used by man for his engineering works since prior to the beginnings of recorded history. Virtually all construction is done on, in, or with soil, but not always are the natural soil conditions adequate to accomplish the work at hand. The basic concepts of soil improvement—densification, cementation, reinforcement, drainage, drying, and heating—were developed hundreds or thousands of years ago and remain unchanged today (ASCE 1978).

While roadway foundations and fortifications have been constructed out of soils for centuries, the development of machines greatly increased the efficiency of such construction. It was the invention of machines in the Industrial Revolution and the 19th century that allowed very significant improvements in the quality and quantity of work undertaken. Drainage methods to improve road performance on poor ground conditions, including crowning transverse grades to drain water away from roadbeds and the use of clean, free draining aggregate to permit the free drainage of water began to be used. In the 20th century, the development of soil mechanics as a discipline provided the basis for understanding soil behavior. This development of improved understanding combined with the development of equipment led to a number of improvement techniques including densification, soil mixing, grouting, and reinforcing methods to mitigate problem ground conditions (ASCE 1978).

Many of the ground modification techniques originated in Europe and the Far East and were subsequently brought to the United States. Often contractors led the development of the techniques as they wrestled with poor ground conditions and made improvements in equipment to make their efforts more efficient and cost-effective. The contractor led development often meant that the techniques were experience-based and sometimes proprietary. FHWA (DiMillio 1999) noted that throughout their short history, commercial and technological innovations [by contractors] in ground modification technologies have almost always preceded research studies of fundamental performance and the development of engineering guidelines.

FHWA (DiMillio 1999), in summarizing the results of Demonstration Project No. 116, Ground Improvement Methods (Elias et al. 1999), noted that ground improvement techniques were found to provide benefits in the following five major areas:

- Utilization of less costly foundation systems
- Reduction in right-of-way acquisitions
- Less environmental disturbance
- Reduction in construction time
- Improved traffic control through construction zones

The impetus for ground modification has been both the increasing need to use marginal sites for new construction purposes and to mitigate risk of failure or potential poor performance. During the past several decades, ground modification has come of age and reached a high level of acceptance in the geotechnical community. Its use is now routinely considered on most projects where poor or unstable soils are encountered, especially on sites underlain by suspect or uncontrolled fills. From the geotechnical design engineer's point of view, ground modification means the modification of the relevant engineering property (e.g., increase in soil shear strength, reduction of soil compressibility, and reduction of soil permeability) – or the transfer of load to more competent support layers. From the contractor’s point of view, ground modification may mean a reduction in construction time and/or a reduction in construction costs. Both points of view are valid reasons to consider the use of ground modification techniques and are often mutually inclusive.

1.3 Focus and Scope

One of the major tasks within geotechnical engineering is to design, implement and evaluate ground modification schemes for infrastructure projects. During the last forty years significant new technologies and methods have been developed and implemented to assist the geotechnical specialist in providing cost-effective solutions for construction on marginal or difficult sites. The purpose of this manual is to introduce available ground modification methods and applications to design generalists (i.e., project planners, roadway designers, consultant reviewers, etc.), design specialists (i.e., geotechnical, structural, and pavement), construction engineers, and specification and contracting specialists involved with projects having problematic site conditions. This publication was prepared with the practicing transportation specialist in mind and with the benefit of extensive industry review.

This chapter provides a description, history, functions, and categories of ground modification. Additionally, the role of ground modification in addressing project risks and constraints and risk mitigation, and contracting mechanisms and their impact on selection of ground modification technologies are described. Typical unit costs are provided. This chapter also includes description of the web-based *GeoTechTools* (http://www.geotechtools.org) technology selection guidance system and geotechnology catalog, and its use for the initial screening process of developing a short-list of technologies applicable to projects. A discussion of final project-specific technology selection that extends beyond the initial screening that can be developed within *GeoTechTools* is included. Through incorporation of technology and project specific factors, a 12-step process is described that leads to a final selection of a preferred project-specific technology.

The introductory chapter is followed by stand-alone technical category chapters. Each category chapter includes a broad introduction to the technical category including typical applications, a listing of common technologies used in the U.S., and summaries for specific technologies in the category. Each technology summary includes: description; advantages and limitations; applicability; complementary technologies; construction methods and materials; photographs; design guidance; quality assurance methods; costs; specifications; and reference list. Each technical category and the technology summaries therein reflect current practice in design, construction, contracting methods, and quality procedures. Current transportation case histories are included for selected technologies. The nine technical categories and the individual technologies included in each category are shown in Table 1-1.

Table 1-1. Technical Categories and Technology Summaries

Chapter	Category	Technologies
1	Introduction	• All
2	Vertical Drains and Accelerated Consolidation	• Prefabricated Vertical Drains (PVDs), with and without fill preloading
3	Lightweight Fills	• Granular Fills: Wood Fiber; Blast Furnace Slag; Fly Ash; Boiler Slag; Expanded Shale, Clay and Slate, Shredded Tires • Compressive Strength Fills: Geofoam; Foamed Concrete
4	Deep Compaction	• Deep Dynamic Compaction • Vibro-Compaction
5	Aggregate Columns	• Stone Columns • Rammed Aggregate Piers
6	Column Supported Embankments	• Column Supported Embankments • Reinforced Soil Load Transfer Platform • Columns: Non-compressible • Columns: Compressible
7	Soil Mixing	• Deep Mixing • Mass Mixing
8	Grouting	• Chemical (Permeation) Grouting • Compaction Grouting • Bulk Void Filling • Slabjacking • Jet Grouting • Rock Fissure Grouting
9	Pavement Support Stabilization	• Mechanical Stabilization • Chemical Stabilization • Moisture Control
10	Reinforced Soil Structures	• Reinforced Embankments • Reinforced Soil Walls • Reinforced Soil Slopes • Soil Nailing

Considerations that are essential in the selection, design, construction, validation, and monitoring of technologies on any successful ground modification project are listed and discussed in the following sections.

2.0 BASIC FUNCTIONS OF GROUND MODIFICATION

2.1 Typical Functions and Typical Applications

Many ground modification and geoconstruction technologies are available to improve the properties of soils, and the methods can be categorized in a number of ways. Mitchell (1981) provided the following categories in his State-of-the-Art paper: compaction, with emphasis on in-situ deep densification of cohesionless soils; consolidation by preloading and/or vertical drains and electro-osmosis; grouting; soil stabilization using admixtures and by ion exchange; thermal stabilization; and reinforcement of soil. More recently, Munfakh and Wyllie (2000) suggested eight main categories: Densification, Consolidation, Weight Reduction, Reinforcement, Chemical Treatment, Thermal Stabilization, Electrotreatment, and Biotechnical stabilization. The current International Society for Soil Mechanics and Geotechnical Engineering (ISSMGE) technical committee on ground improvement (TC 211, formerly TC17) lists five categories: improvement without admixtures in non-cohesive soils; improvement without admixtures in cohesive soils; improvement with admixtures or inclusions; improvement with grouting type admixtures; and earth reinforcement (Chu et al. 2009). Herein ground modification technologies are categorized by the functions introduced in Section 1.1 and are discussed in more detail below.

Within transportation infrastructure a number of applications can benefit from the use of ground modification. The applications include bridge support, embankments, embankment widening, pavement support, and construction working platforms. Each of these applications is discussed in more detail below.

2.1.1 Functions

2.1.1.1 Increase Shear Strength and Bearing Resistance

Here the function of the ground modification is to increase the soil's strength, which in turn increases the bearing resistance for foundations and embankments. Increases in soil strength and bearing resistance can be accomplished by densifying loose cohesionless soils, consolidating soft clay soils, or the addition of cementing agents to the soil.

2.1.1.2 Increase density

This function generally applies to the densification of loose sands through technologies that add energy to the soil through a vibration or dynamic process. The imparted energy changes the loose sand into a more dense state. The more dense soil has increased strength and bearing resistance and increased resistance to liquefaction. In cohesive soils increasing the density is accomplished through consolidation processes that remove water from the void spaces thus reducing the amount of settlement that will occur when loads are applied to the soil.

2.1.1.3 Decrease Permeability

Here the function is to decrease the amount of water flowing through the soil. This can be accomplished by increasing the density of the soil or through the addition of grouts or binders that make the soil relatively impermeable or fill fissures.

2.1.1.4 Control Deformation

Controlling deformation includes reducing total settlement, heave, and distortion caused by differential settlement. Methods include those that densify or consolidate the foundations soils, or strengthen the soils through grouts or binders to control deformations. Deformation control can also be accomplished through the use of columns to transfer loads to more competent materials. Expansive soil heave can be treated using binders that mitigate the effects of water.

2.1.1.5 Increase Drainage

Increasing drainage allows for more efficient removal of water from foundation soils, subgrades, and base and subbase courses. Almost all soils are improved in their strength and stiffness properties with reduction in water. Increased drainage can also be used to reduce liquefaction susceptibility of cohesionless materials.

2.1.1.6 Accelerate Consolidation

Accelerating consolidation reduces the time involved for settlement in foundation soils to occur. Consolidation can be accelerated by reducing the drainage path length for cohesive soils in combination with embankment loading or fill preloading. This can be accomplished through the use of prefabricated vertical drains or other columns that allow water an easier flow path.

2.1.1.7 Decrease Imposed Loads

Decreasing imposed loads through the use of lighter weight fill materials reduces loads on weak soils reducing settlement and stability issues.

2.1.1.8 Provide Lateral Stability

Change in grade requirements can be accomplished by use of a number of earth retaining systems that provide lateral support and stability to site soils, in both cut and fill situations. Such support can be provided for both vertical and sloping cases.

2.1.1.9 Increase Resistance to Liquefaction

The resistance of cohesionless soils to liquefaction can be accomplished by densifying the soils through vibratory or dynamic methods that increase the density of the cohesionless materials. Other means of increasing resistance to liquefaction include the addition of grouts and binders to the soil matrix, increased drainage of the soils, and isolation of the potentially liquefiable soils.

2.1.1.10 Transfer Vertical Loads to More Competent Soil or Rock Layers

Here, vertical loads – typically embankments or fill retaining structures – are transferred through loose or weak soils by columns that transfer the embankment loads to more competent layers. This technique helps control settlement, particularly differential settlement, and stability of the highway feature on the unstable soils.

2.1.2 Applications

Ground modification and geoconstruction technologies can be used in a number of highway and transportation infrastructure applications. Common applications are discussed below.

2.1.2.1 Structure Support

Bridges are used to cross water and also to provide grade separation pathways over other highways, railroads and other infrastructure. At water crossings, the bridge is often situated in an alluvial environment in which nature has left a variety of soil deposits ranging from soft clays to loose sands to dense sands. The deposits are often interlayered and non-uniform. While deep foundations are often used to support bridge abutments and piers, ground modification technologies are alternatives that can be used to improve the site conditions, allowing less expensive shallow or intermediate foundations to be used. Retaining structures can also be used for abutments and approach embankments for bridges for both water crossings and grade changes.

2.1.2.2 Embankments

Embankments are used to support highways. They are used to change the grade along an alignment to provide better vertical position for the roadway and as approaches to bridges.

Embankments are often constructed across unstable soils. Ground modification technologies can be used to improve the unstable foundation soils to reduce settlement and stability issues, while avoiding excavation and disposal of the unsuitable soil.

2.1.2.3 Embankment Widening

The need to increase capacity of roadways often means adding lanes of traffic to existing roads. In locations where the existing roadway is constructed on compressible unstable soils, widening the road by adding embankment can lead to differential settlement between the existing and the new embankments, global instability, etc. Ground modification can stabilize the compressible or unstable soils, reducing the potential for the unwanted movements. This can be accomplished using lightweight fill materials, column supported embankments, and methods that increase the bearing resistance of the underlying soils.

2.1.2.4 Pavement Support

The pavement section is supported by the subgrade soil, which in some cases is poor, requiring very thick structural sections, and may not even support construction equipment. Support for the pavement section can be increased in several ways including stabilizing the subgrade, and using alternative or recycled materials. The subgrade and base layers can be improved through mechanical and chemical means to improve strength, and also by drainage efforts to reduce the adverse effects of water.

2.1.2.5 Construction Working Platform

Construction platforms are almost always needed to support ground modification equipment on poor soils that are being stabilized. They are also often needed for temporary roadways to allow construction to proceed and for storage of equipment and materials during construction. The means of support for working platforms is similar to that for pavement support, but is considered temporary in nature.

2.2 Applicability Limits

All ground modification technologies have limits on their applicability. Limitations may be defined as soil type applicability, depth of treatment, etc. General applicability of the technologies covered within this manual is summarized in Table 1-2, by technology category. The advantages and potential disadvantages of each technology are listed and discussed under their respective category chapter.

Table 1-2. General Applicability of Technologies

Category	Technologies	Applicability
Vertical Drains and Accelerated Consolidation	PVDs, with and without fill preloading	Compressible clays, saturated low strength clays
Lightweight Fills	Compressive Strength Fills: Geofoam; Foamed Concrete	Broad applicability; no geologic or geometric limitations
Lightweight Fills	Granular Fills: Wood Fiber; Blast Furnace Slag; Fly Ash; Boiler Slag; Expanded Shale, Clay, and Slate; Tire Shreds	Broad applicability; no geologic or geometric limitations
Deep Compaction	Deep Dynamic Compaction	Loose pervious and semi-pervious soils with fines contents less than 15%, materials containing large voids, spoils and waste areas
Deep Compaction	Vibro-Compaction	Cohesionless soils, clean sands with less than 15% silts and/or less than 2% clay
Aggregate Columns	Stone Columns	Clays, silts, loose silty sands, and uncompacted fill
Aggregate Columns	Rammed Aggregate Piers	Clays, silts, loose silty sands, uncompacted fill
Column Supported Embankments	Column Supported Embankments	Soft compressible clay, peats, and organic soils where settlement and global stability are concerns
Column Supported Embankments	Reinforced Soil Load Transfer Platform	Soft compressible clay, peats, and organic soils where settlement and global stability are concerns
Column Supported Embankments	Columns: Non-compressible	All soil types, in particular weak soils that cannot support surface loads
Column Supported Embankments	Columns: Compressible	All soil types except very soft soils low undrained shear strength
Soil Mixing	Deep Mixing	Suitable in large range of soils, ones that can be stabilized with cement, lime, slag, or other binders
Soil Mixing	Mass Mixing	Peat, soft clay, dredged soil, soft silt, sludges, contaminated soils
Grouting	Chemical (Permeation) Grouting	Wide range of soil types including weakly cemented rock-fill materials

Category	Technologies	Applicability
Grouting	Compaction Grouting	Cohesionless granular soils, collapsible soils, and unsaturated fine grained soils; may be used to fill voids in sinkholes or abandoned mine shafts; can arrest settlement under a structure and lift foundations that have settled
Grouting	Jet Grouting	Wide range of soil types and groundwater conditions
Grouting	Rock Fissure Grouting	Structural stability and groundwater control
Grouting	Bulk Void Filling and Slabjacking	All soil types were voids develop under pavements
Pavement Support Stabilization	Mechanical Stabilization	Weak subgrades, loose sands, and to stabilize thin aggregate layers on subgrades with CBR<8
Pavement Support Stabilization	Chemical Stabilization	Portland cement and lime: high plasticity clays Fly ash: soils with little or no plastic fines Asphalt: silty, sandy and granular soils Cement Kiln Dust: expansive soils
Pavement Support Stabilization	Moisture Control	All soil types
Reinforced Soil Structures	Reinforced Embankments Over Soft Soils	Soft soil foundations, with no limitation on depth of soft soils
Reinforced Soil Structures	Reinforced Soil Walls	Well suited in fill embankments, steep-sided terrain, ground subject to soil instability and where foundations soils are poor
Reinforced Soil Structures	Reinforced Soil Slopes	Can be constructed over any firm foundation
Reinforced Soil Structures	Soil Nail Walls	Dense to very dense granular soils with apparent cohesion, weathered rock, stiff to hard fine-grained soils, engineered fill, residual soils, glacial till

2.3 Feasibility Evaluations

The feasibility of a ground modification method for a particular project need depends upon the function(s) of the modification and the method(s) selected to carry out the function.

Feasibility evaluation includes the identification and evaluation of: technical issues, project development/delivery methods, performance criteria and quality assurance procedures, and non-technical issues that affect the utilization of ground improvement and geoconstruction technologies. A generalized summary of the evaluation process for use with the technologies in this reference manual can be summarized as:

1. Identify project conditions which could require ground modification or geoconstruction technologies, such as projects that encounter:
    a. Poor ground conditions which will not provide adequate support for the transportation related structure. Poor ground conditions are typically characterized by soft or loose foundation soils, which, under load, would cause long-term vertical and/or lateral deformations, or cause construction or post-construction instability.
    b. Project constraints which require retaining walls or steep slopes.
    c. Pavement foundations which require improvement.
    d. Need for of a working platform or access road.
2. Identify or establish performance requirements.
3. Identify and assess any time, space or environmental constraints.
4. Assess the site conditions.
5. Assess project constraints.
6. Identify limitations on the use of ground modification technologies.
7. Consider alternatives to the use of ground modification technologies.

These items are discussed in more detail below.

2.3.1 Project Constraints

Project constraints are generally such items as the project schedule and time, phasing requirements; budget and cost; project conditions such as right-of-way limits, geometry, scale, utilities and sequence; traffic flow/interruption and congestion; weather; and environmental factors. Schedule acceleration is of key importance on almost all projects today. Therefore speed of ground modification construction is often a key selection factor.

2.3.2 Geotechnical Performance Criteria/Indicators

For each project, performance requirements need to be established. These can be thought of as the operational criteria for the facility. The performance criteria/indicators are typically defined in terms of geotechnical performance criteria such as stability requirements, allowable long-term deformations (total or differential settlement), rate of settlement, seepage (quantity and flow rate), and durability (service life) and maintenance activities. These criteria establish the level of improvement required in terms of soil properties such as strength, density, modulus, compressibility, and hydraulic conductivity.

2.3.3 Environmental and Space Considerations

Environmental and space constraints must be identified and assessed. Environmental constraints may include the disposal of spoils from the particular ground modification technology, the disposal of waste materials encountered on the site, protection of the site from erosion, protection of surface and ground waters from pollution, and the effects of construction vibrations, noise and dust. Space constraints typically refer to site accessibility, sufficient area for construction equipment to operate safety, overhead clearance, and adjacent structures and utilities.

2.3.4 Site Conditions

Site conditions, first and foremost, means assessment of the subsurface conditions. The level of detail regarding the assessment of subsurface conditions will vary significantly across the wide range of transportation related projects and the type of ground modification selected. Regardless of the project type, the soils which will affect the performance requirements must be identified and the necessary engineering properties established to perform a preliminary design for the project. At a minimum, the type, depth, and extent of needed treatment must be determined, as well as the location of the groundwater table. For sites with poor ground conditions, it is also valuable to have at least a preliminary assessment of the shear strength, compressibility, and organic content of the identified poor soils. Additionally, assessment of subsurface obstructions in terms of cobbles, boulders, or construction debris, water bearing sands, organic layers, and very stiff surface deposits can affect the selection of appropriate technologies. The availability of materials for construction such as sand, gravel, and water are also important site considerations.

2.4 Limitations

Limitations that can affect the use of ground modification technologies include those that are general in nature, those that are related to the site, and those that are technology dependent.

A number of non-technical and project specific parameters can limit the use of ground modification technologies. General limitations include items such as lack of knowledge about technologies; lack of organizational structure and policies to encourage use of innovations; absence of simple, comprehensive, reliable analysis and design procedures; lack of effective quality assurance procedures; undefined established engineering parameters and/or performance criteria; long-term performance uncertainty; and availability of equipment, experienced personnel, and appropriate materials. These limitations can be overcome in a number of ways, including promotion of technologies, collaboration, education efforts, development of agency ground modification technology champions, documentation of agency case histories, and hosting demonstration projects.

Limitations related to the site include a lack of site characterization information; environmental constraints; obstructions, both above the ground and below the ground; development within the immediate vicinity; unusual construction loads and vibrations; and environmental conditions. These limitations will vary for each site and for specific technologies and need to be assessed on a project-specific basis.

A general, potential limitation on the use of ground modification methods is that most techniques have been traditionally designed using allowable stress design (ASD) rather than the recently implemented load and resistance factor design (LRFD). Currently, most ground modification methods have not been calibrated as part of the LRFD conversion process that was applied to structural foundations and selected (i.e., MSE walls, soil nail walls) earth retention methods. Many methods do not lend themselves to an LRFD approach. Additionally, some ground modification methods are proprietary and/or have black box design methods.

In addition to general and site limitations, each technology has limitations on its use such as headroom requirements, depth limitations, design and quality assurance requirements, time to construct and to be effective, and so on. The limitations for specific technologies are discussed in more detail in the technology summaries.

2.5 Alternative Solutions

The use of a ground modification technology is not the only feasible means of addressing poor ground conditions and alternative solutions should always be considered. Alternatives include bypassing the poor ground conditions through relocation of the project to a more suitable site or through the use of a deep foundation; removing and replacing the unsuitable soils; designing the planned structure to accommodate the poor/marginal ground conditions; or using a structural solution such as a bridge.

3.0 TECHNOLOGY CLASSIFICATION AND ELEMENTS

3.1 Classification by Function

As noted in Section 1.1, ground modification solutions can provide a number of functions and ground modification technologies can be classified in a variety of ways. Herein, technologies are classified by the functions defined in Sections 1.1 and 2.1.1. Table 1-3 shows the technologies included in this manual classified by function. As can be seen, there is considerable overlap as many technologies fit into several functional categories.

Table 1-3. Technologies Classified by Function

Function	Technologies	Comment
Increase shear strength and bearing resistance	• Vibro-Compaction • Dynamic Compaction • Compaction Grouting • Mixing Methods • PVDs • Stone Columns • Rammed Aggregate Piers • Chemical Stabilization • Mechanical Stabilization	Some technologies will work in all soil types; others are limited to cohesive or cohesionless soils.
Increase density	• Vibro-Compaction • Dynamic Compaction • Blasting Compaction • Compaction Grouting • Mixing Methods • PVDs	Some technologies will work on all soil types; others are limited to cohesive or to cohesionless soils.
Decrease permeability	• Bulk-infill Grouting • Chemical Grouting • Jet Grouting • Deep Mixing Methods	Type of grouting dependent upon soils, depths, geology, and design requirements.
Control deformations (settlement, heave, distortions)	• Columns Supported Embankments • Reinforced Load Transfer Platforms • Non-compressible Columns • Mixing Methods • Vibro-Compaction • Dynamic Compaction • Stone Columns • Rammed Aggregate Piers • Chemical Stabilization • Mechanical Stabilization • Encapsulation	Technologies generally used to bypass or isolate soft ground or to modify and improve the soft ground.

Function	Technologies	Comment
Increase drainage	• PVDs • Aggregate Columns • Geotextile Encased columns • Electro-osmosis • Geosynthetics in Pavement Drainage	Generally increase drainage by inserting a drainage path within the soil to be drained.
Accelerate consolidation	• PVDs • Aggregate Columns • Geotextile Encased Columns	Acceleration due to decreased length of flow path to dissipate excess pore water pressure.
Decrease imposed loads	• Granular Fills (Wood Fiber; Blast Furnace Slag; Fly Ash; Boiler Slag; Expanded Shale, Clay & Slate; Tire Shreds) • Compressive Strength Fills: (Geofoam, Foamed Concrete)	Densities vary from 1 to 90 pcf. Granular fill usage subject to local availability.
Provide lateral stability	• Mechanically Stabilized Earth (MSE) Walls • Reinforced Soil Slopes • Soil Nailing	Techniques based on internal reinforcement of soils.
Increase resistance to liquefaction	• Aggregate Columns • Deep Dynamic Compaction • Deep Mixing • Jet Grouting • Vibro-Compaction	Techniques generally to modify and improve soil susceptible to liquefaction. Aggregate columns also provide drainage path for excess pore water pressure.
Transfer embankment loads to more competent layers	• Column Supported Embankments • Reinforced Soil Load Transfer Platforms • Non-compressible Columns • Compressible Columns	A column supported embankment may be constructed on a load transfer platform that is supported on columns.

Within the *GeoTechTools* system, technologies are classified by applications for a variety of geotechnical solutions. Thus within *GeoTechTools*, lists of technologies are provided under the following geotechnical solutions: earthwork construction, soft ground drainage and consolidation, densification of cohesionless soils, construction of vertical support elements, embankments over soft soils, lateral earth support, cutoff walls, liquefaction mitigation, increased pavement performance, void filling, and sustainability.

3.2 Elements of Construction

Difficult, unstable ground conditions requiring ground modification solutions are typically characterized by soft or loose foundation soils, which under load, would cause excessive short- or long-term settlement or cause construction or post-construction instability, The technologies discussed in this manual are generally applicable to elements of construction over , construction over stable or stabilized soils, geotechnical pavement components, and working platforms. These elements are shown in Figure 1-1 and are further discussed below. These elements are often used in conjunction with structures such as bridges.

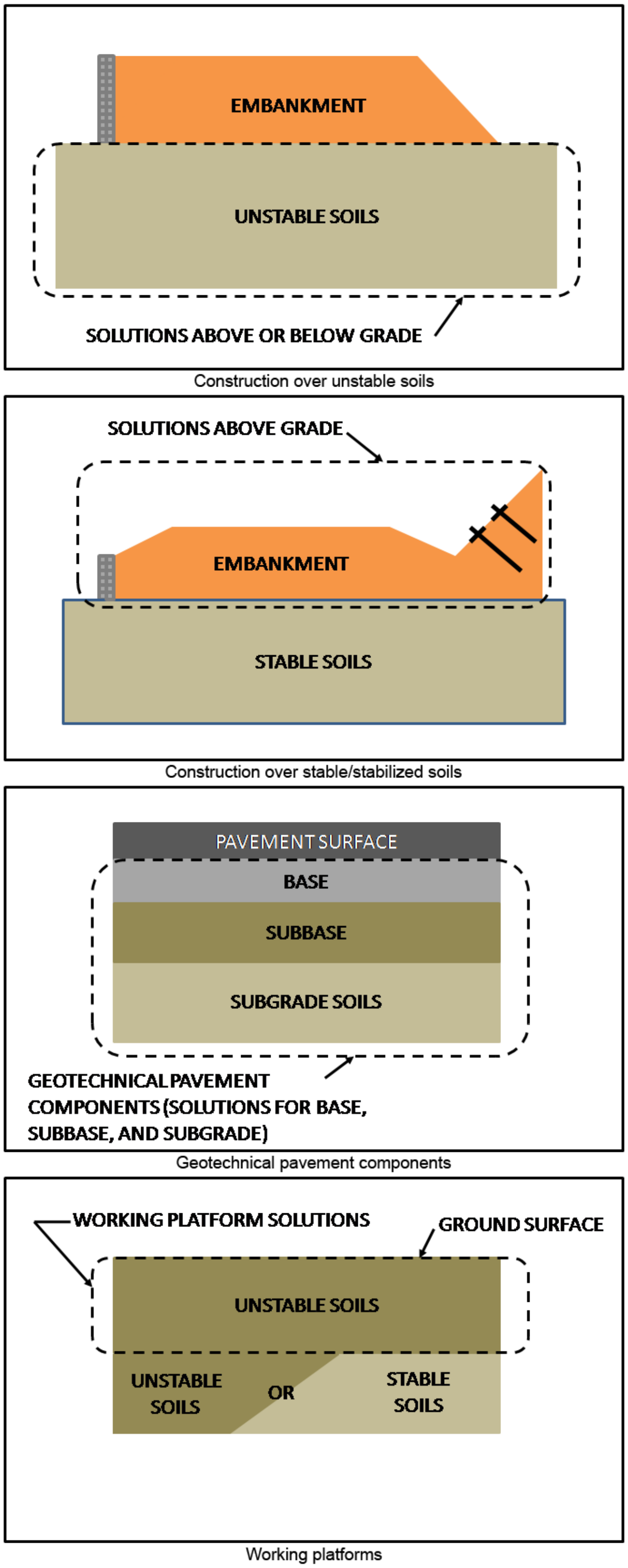


Figure 1-1. Elements of construction.

Construction over Unstable Soils focuses on methods to support embankment and embankment widening on the weak or loose foundation soils, i.e., typically below-grade technologies. Methods include ground modification technologies that improve the unstable soils so that the embankments can be safely constructed without unacceptable deformation issues. Although the ground modification is often below-grade, some at-grade technologies are also applicable to this element. Applications include bridge abutment support, approach embankments, roadway widening, and support of structural elements.

Construction over Stable or Stabilized Soils focuses on methods for embankment and/or embankment widening construction, i.e., above-grade technologies. Methods include fill placement and compaction procedures, reduction of embankment width/volume, fill earth retention systems, and slope stabilization systems. The ground modification methods strengthen the embankment materials, and allow for geometric constraints such as retaining walls, or stabilizing cut slopes. Applications include retaining systems for bridge abutments and to reduce right-of-way needs.

Geotechnical Pavement Components focuses on methods to improve the supporting elements of pavements: the subgrade, and base and subbase courses. Methods include fill placement, stabilization, grouting, and reinforcement technologies. Recycling/reuse of materials in the geotechnical pavement support sections are also included.

Working Platforms focuses on methods to provide working platforms on either stable or unstable subgrades. Methods include fill placement, stabilization, and reinforcement technologies. Recycling/reuse of materials in the geotechnical pavement support sections are also included. Working platforms are also applicable to Construction over Unstable Soils and Geotechnical Pavement Components. Working Platforms are generally assumed to be temporary structures.

4.0 CONSTRAINTS AND RISK MANAGEMENT

Constraints are limitations or restrictions on activities and can affect project delivery and performance. Constraints are often project specific, and can also pose risks to the successful completion and performance of a project. Such risks can be identified and managed through a risk management process. Ground modification methods can be used to mitigate some specific project risks, and can be selected to work within a given project set of constraints. A discussion of constraints and risk management follows.

4.1 Types of Constraints

Constraints are often driven by factors external to the project owner. Project constraints vary by project and could include general or specific issues and restrictions in the following areas: economic, legal, environmental, social, management, time, and technical. Each of these areas might include a related list of subtopics which would be addressed collectively or separately. Herein, with the emphasis on the use and application of ground modification technologies, constraints are defined in two categories, general constraints and geotechnical constraints. The key to constraint mitigation is the early identification and clear definition of the constraint.

4.1.1 General

General constraints include all those items not explicitly connected with the technical (or geotechnical) aspects of the project. For transportation infrastructure projects key general constraints include the project schedule and time; budget and cost; project conditions such as right-of-way limits, geometry, scale, utilities and sequence; traffic flow/interruption; weather; environmental; availability of agency personnel, agency organization and structure, project management philosophy; contracting processes; public perceptions; and liability. Some of these constraints influence the project during its entire life while others become important at some point during project development or during and following construction. In general it is better to identify constraints as early in a project as possible so that a proactive approach can be taken to address and resolve the constraint. Many constraints can be complex, for example, time can include the time consideration related to planning, for design, for procurement, and the time available for construction. Depending on the overall project schedule and critical milestones a delay in one project phase may affect the time of the overall project.

4.1.2 Geotechnical

The focus of the geotechnical constraints are those specific to geotechnical aspects of the project and the ground modification technologies that might be used. Such constraints include subsurface conditions, including archaeological remains, contaminated groundwater, obstructions, and buried utilities; proprietary products/processes; knowledge and experience with specific technologies; allowable movements-both vertical and lateral; stability requirements; noise and vibrations due to construction equipment; time for modification to be effective (some improvement methods occur over a defined time period); spoils development, volume, and disposal; environmental impacts of the technology; availability of qualified contractors, personnel, materials and specialty equipment; and constructability of the selected technology.

4.2 Types of Risks

In engineering contexts, risk is the product of probability of an event and its consequences (Baecher and Christian 2003). The expectation is that the consequence is an adverse outcome. As applied to geotechnical construction, risk might be considered as the probability that an adverse event will occur and that the event will impact achievement of project objectives. Hence the geotechnical risks associated with a project are those in which an adverse geotechnical outcome occurs that has an impact on the project, whether that impact is simply a delay in completion or perhaps the redesign of a geotechnical component (e.g., foundation) to accommodate unforeseen conditions. A key geotechnical risk is the failure to identify potential problems and design for their consequences. Possible geotechnical risks may relate to the following:

- Slope Instability – soil and rock slopes (natural and man-made)
- Deformations (vertical and lateral)
- Bearing capacity
- Liquefaction
- Subsidence (sub-surface voids or change in subsurface stresses)
- Groundwater levels and fluctuations
- Chemically reactive ground
- Contamination
- Unforeseen ground conditions (extent, type) [or from a contractor’s point of view: differing geotechnical site conditions]
- Problem ground conditions – expansive, collapsible, organic/peat, and dispersive soils, fills, landfills
- Inadequate geotechnical investigation

- Inadequate design
- Poor construction or inadequate construction monitoring

Additionally, one could consider risks associated with material characterization (properties), geotechnical models, service conditions, load calculations, resistance quantification, as well as risks related to project development, planning, and execution. The importance of considering geotechnical risks is highlighted by the fact that contract disputes in highway works are often related to geotechnical issues.

Geotechnical risks can be accepted, managed, and mitigated (Chapman 2012). Indeed an entire specialty area has developed around risk management, as discussed in the next section. Sound geotechnical practices related to design, construction, and monitoring, such as hazard recognition during site investigations can substantially reduce risks.

The magnitude of transportation infrastructure construction is very large and increasingly complex. A number of delivery/contracting methods are being used to construct transportation facilities including Design-Bid-Build, Design-Build, Public Private Partnership (P3), and Construction Manager-General Contractor (CMGC). Geotechnical risks can occur in all these types of construction/delivery contracting methods; the risks remain but are shifted among the participants. Geotechnical risks in Design-Build (D-B) construction contracts was the subject of a recent TRB Synthesis (Gransberg and Loulakis 2012) due to the large increase in D-B contracts in transportation work. The study looked at the differences in geotechnical scope risk, geotechnical cost risk, and geotechnical schedule risk between design-bid-build (D-B-B) and design-build (D-B) contracts. Risks in scope, cost, and schedule are key factors in a contractor's successful completion of a project. Table 1-4 summarizes the results of the study.

Table 1-4. Design-Bid-Build (D-B-B) versus Design-Build (D-B) Risk Profiles

Contract Type	Risk Type	Contractor Team	Owner
D-B-B	Scope	• Warranties and Guarantees • Latent Defects - Workmanship • Competent Geotechnical Construction Personnel Available	• Design Error and Omissions • Latent Defects - Design • Direct and Tacit Approval of Constructive Changes to Design
D-B	Scope	• Design Errors and Omissions • Warranties and Guarantees • Latent Defects – Design and Workmanship • Competent Geotechnical Design Personnel Available	• Clear Geotechnical Scope Definition • Direct and Tacit Approval of Constructive Changes to Geotechnical Design • Geotechnical Design Review Comments and Directives • Technical Review Capability
D-B-B	Cost	• Rework • Subcontractor Default • Market Fluctuation after Award	• Redesign and Resultant Rework • Construction Contract Amount • Market Fluctuation During Design - Material and Labor
D-B	Cost	• Rework • Redesign • Subcontractor Default • Market Fluctuation During Design - Material and Labor	• Design-Build Contract Amount • Prompt Payment • Design-Builder Default
D-B-B	Schedule	• Contract Completion • Date • Liquidated Damages	• Timely Design Completion • Owner Furnished Property Delivery
D-B	Schedule	• Delivery on Approved Schedule • Fast-Track Geotechnical Rework • Liquidated Damages	• Unrealistic Schedule • Timely Geotechnical Design Approvals on Fast-Track Project • Owner Furnished Property Delivery

Source: Gransberg and Loukakis 2012

Gransberg and Loulakis (2012) make clear that the risks don't go away, but rather responsibility for some risks may shift. For example, they note how the items design errors and omissions, latent defects in design, and market fluctuations during design for materials and labor move from the Owner column in D-B-B to the Design-Builder column in D-B.

4.3 Risk Management Process

The risk management process can be defined as "The systematic application of management policies, procedures and practices to the activities of communicating, consulting, establishing the context, and identifying, analyzing, evaluating, treating, monitoring and reviewing risk." Risk management is viewed as a way to focus limited resources, strengthen the ability to efficiently manage program delivery, and improve communication and manage risk corporately.

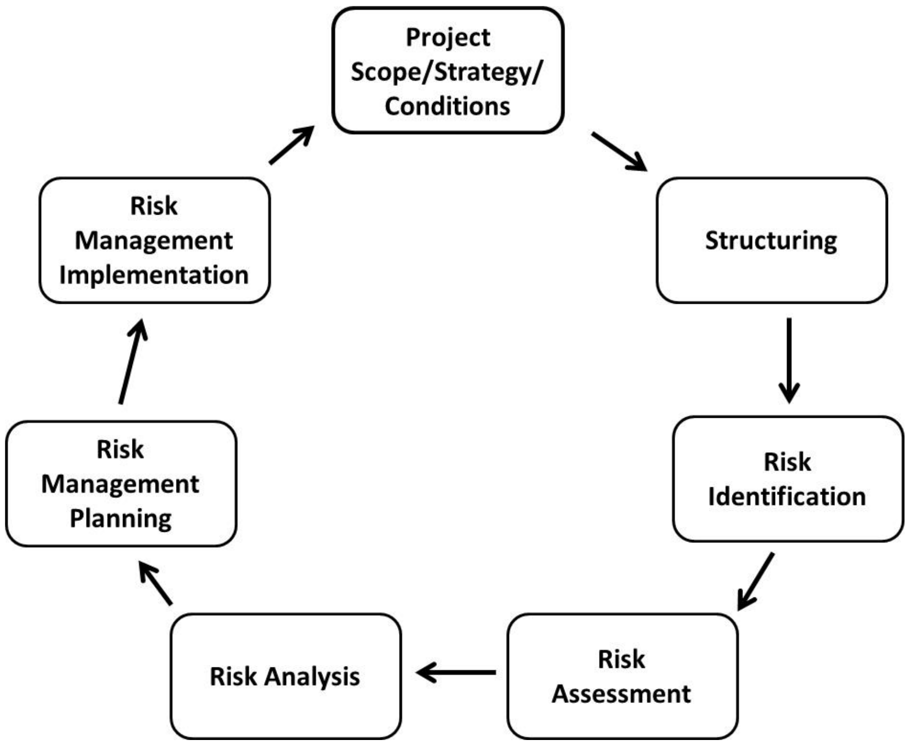
The first part of the risk management process is the identification of the context of the risk, identifying project objectives, the criteria on which to assess the risks and who will do the assessment. A major portion of this activity is risk assessment in which the risks are identified, analyzed and assessed, and prioritized in a risk register. Risk response plans are then developed. A risk tracking plan is then developed which allows monitoring, evaluation, and adjustment as responses to risk are made and new risks are included. This process can be applied at many levels including at project, program and enterprise levels. Unfortunately, decision making for ground modification technologies has often been reactive rather than proactive. Use of the risk management process applied to geotechnical aspects of a project can help identify potential events that might affect a project and allow for proactive use of the technologies discussed herein. Note that communication and consultation occur at each and every step, starting at the very beginning with identification and engagement of stakeholders.

A recent SHRP2 study on the Process for Managing Risk on Rapid Renewal Projects (R09) is a significant resource on risk management for all types of projects, with particular emphasis on geotechnical issues (Golder Associates Inc., 2011). As noted within this project report:

> The primary objective of the risk management process, whether at the individual project level or for a "program" of individual projects, is to optimize project performance (e.g., minimize cost, minimize disruption, etc.). Problems can arise during a project that leads to undesirable performance. Anticipating the problems upfront can lead to management strategies that minimize undesirable performance. An appropriate formal risk management approach also needs to be efficient and defensible, as well as adequate (as

opposed to perfect), in achieving the objective of optimizing project performance. The process also must be compatible with the DOT organization and their projects.

The risk management process can be conducted in a number of ways. The SHPR 2 R09 process is shown in Figure 1-2 and consists of six steps which form a continuous loop, indicating a continual process of communication and updating. Each step is briefly summarized below.



After Golder Associates Inc. 2011

Figure 1-2. SHRP 2 R09 iterative risk management process.

To start the process, the project scope, strategy and conditions must be identified and enumerated. The first step in the risk management process is Structuring, which is where base project performance is identified and includes the planned project scope, strategy, and key conditions and assumptions. A base project description is developed and includes all the relevant elements of the project. The second step is Risk Identification where the risks and opportunities relative to the base project performance are enumerated. The objectives of risk identification are to: identify, categorize, and document all risks and opportunities that could significantly affect the project's base performance measures; start a risk register, which is a comprehensive set of non-overlapping risks and opportunities; and set the stage for subsequent steps in the risk-management process. The third step is Risk Assessment where the relative severity of the risks and opportunities are assessed so they can be prioritized for subsequent management. As the project develops and conditions change, the risk factors for previously identified risks might change and need to be reassessed, while the factors for any new risks must be assessed. The updated assessments of factors describing the severity of each risk are maintained in the project risk register.

The fourth step is Risk Analysis where a more quantitative determination of project performance is assessed using uncertainty and probability distributions. This step is particularly valuable on complex projects where large amounts of information must be processed. In this step the uncertainty in project performance measures are quantified and sensitivity analyses conducted to assess individual risks to the base performance requirements. The fifth step is Risk Management Planning. Here proactive ways to manage the identified risks are identified and evaluated for possible use should the risk develop. As the project proceeds and risks change, these plans must be reviewed and updated as necessary to optimize project performance. Step six, Risk Management Implementation, consists of implementing and monitoring the developed Risk Management Plan. This step insures that the periodic review and updating of all steps is done on a timely and appropriate basis. This step is tied back to the project scope, strategy and conditions identified at the start of the process.

5.0 CONTRACTING ALTERNATIVES, SPECIFICATIONS, AND QUALITY ASSURANCE

Project planning is concerned with understanding project constraints and identifying risks. Constraints and risks will always be present on a project, however, the discernment of constraints and the management of risks can be affected by the contracting methods used. The sections below discuss contract mechanisms, specifications, quality assurance, and instrumentation monitoring and construction control methods to deal with constraints and risks.

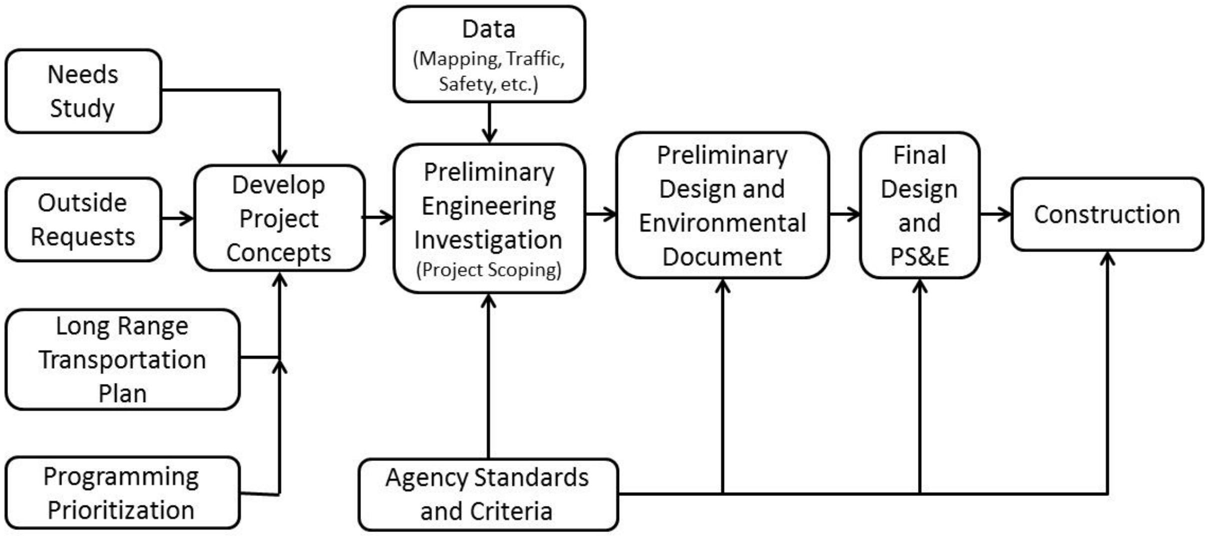
5.1 Contracting Mechanisms Used in Project Delivery

The development and construction of transportation projects involves a number of processes and requires a high level of coordination among many stakeholders. Several contracting mechanisms are available for delivery of projects; however, project delivery extends beyond the procurement phase and approach of a particular project. Herein, *project delivery* refers to the entire process from initial planning and programming to close-out of construction while *contracting or contracting method* refers to the roles and responsibilities for conducting design and construction activities. Transportation agencies play a significant role in the implementation of a project from its inception to its construction, with responsibilities that include: estimating and controlling costs; ensuring the fulfillment of environmental and Federal requirements; obtaining adequate financing and the overall managing of the various parties involved in bringing the project to a successful completion.

While nomenclature may vary from state to state, and activities may overlap, the following items related to project delivery are generally contained somewhere within each state's process.

- Planning and programming
- Conceptual studies
- Environmental stewardship
- Right-of-way
- Preliminary design
- Final design and PS&E (plans, specification, and estimate)
- Construction

Figure 1-3 from the Federal Lands Highway Project Development and Design Manual (PDDM) provides a generalized concept of the project development work process (FHWA 2011).



After FHWA 2011

Figure 1-3. Federal Lands Highway project development work process.

From this figure it can be seen that projects begin from needs studies and outside requests for projects. The projects are then considered and placed in a long-range transportation plan with programming prioritization occurring, leading to the development of project concepts (planning). The boxes on the left hand side of this figure collectively constitute what is generally called planning and programming. What follows next is project scoping which requires the collection of data, adherence to agency standards and criteria, and preliminary engineering studies. The next step is preliminary design. The environmental studies and documentation may occur during project scoping or preliminary design, depending upon the project and the transportation agency. Once preliminary design is completed and approvals are secured, the project can move into final design and development of plans, specifications, and estimates. Next follows construction.

Traditionally, the design and construction phases of this process were conducted using the design-bid-build process wherein the agency (or its consultants) designed the project, the design was let out for bidding, with the lowest responsible bidder awarded the construction contract. In recent decades several other contracting mechanisms have been developed including Design-Build (DB), and Construction Manager-General Contractor (CMGC). Each of these methods has a slightly different approach to the project work process on a project and each has advantages and challenges.

In D-B, the contracting method combines two, usually separate project phases into a single contract, and one entity – the design-build team – works under a single contract with the project owner to provide design and construction services. This provides unified work flow from conceptualization through completion – thereby integrating the roles of designer and constructor. In this method, owners generally execute a single, fixed-fee contract for both engineering services and construction. The D-B entity may be a single firm, a consortium, joint venture or other organization assembled for a particular project.

Public-Private-Partnerships (P3) is contractual agreements formed between a public agency and a private sector entity that allow for greater private sector participation in the delivery and financing of transportation projects. P3 projects also routinely include an operations component for a defined time period and tolls. Following the operating period the P3 project is returned to the original owner in a defined state of good condition. In a P3, a government entity contracts with a private venture to develop, construct, and operate a public project. Often the private entity assumes substantial financial, technical, and operational risk in the project. There are many different P3 structures, and the degree to which the private sector assumes responsibility – including financial risk – differs from one application to another. Additionally, different types of P3s lend themselves to the development of new facilities and others to the operation or expansion of existing assets.

The CMGC process involves the project owner hiring a contractor to provide feedback during the design phase before the start of construction. The CMGC process involves two contract phases. The first contract phase, the design phase, allows the contractor to work with the designer and the project owner to identify risks, provide costs projections and refine the project schedule. Once the design phase is complete, the contractor and project owner negotiate on the price for the construction contract. If all parties are in agreement with costs then the second contract phase, the construction phase, is kicked off and construction begins. This method is often used to accelerate project delivery.

As discussed in Section 4.2, these various contracting methods do not in and of themselves eliminate risks, the risks are simply shifted among the various entities involved.

5.2 Specification Development

Preparation of specifications is an integral part of the design process for the use of any ground modification technology. Specifications are the written instructions describing the work that is to be undertaken. Specifications are part of the contract documents, which also include the drawings, bid or proposal documents, agreement forms, and contract modifications. Specifications communicate to bidders prior to contract award, and to the selected contractor thereafter, the definitive directions, procedures, and material and equipment requirements the owner considers necessary for completing the contract work. In this context, specifications directly affect the quality of design and construction of every highway product, as well as the cost of construction and maintenance (FHWA 2010 and Scott et al. 2014).

Often state DOTs do not have specifications or special provisions prepared for ground modification technologies. Or if they do, it is only for a limited number of ground modification technologies that have been previously used by that agency. Hence, specifications will often have to be developed to use ground modification methods on a particular project. Guidance on specification development for particular technologies is covered in the category chapters for individual technologies in this manual. Guide specifications for specific technologies can be found in *GeoTechTools* system in the individual technology information pages (see Section 7). These guide specifications reflect a summary of existing specifications for technologies and, in many cases, provide a preferred guide specification for the specific technology that can be used in specification development for application to specific projects.

5.3 Quality Assurance

TRB Circular E-C037 (TRB 2002) defines Quality Assurance (QA) as "All those planned and systematic actions necessary to provide confidence that a product or facility will perform satisfactorily in service." The circular goes on to state the following:

> QA addresses the overall problem of obtaining the quality of a service, product, or facility in the most efficient, economical, and satisfactory manner possible. Within this broad context, QA involves continued evaluation of the activities of planning, design, development of plans and specifications, advertising and awarding of contracts, construction, and maintenance, and the interactions of these activities.

For many years the term QC/QA was used to describe quality activities associated with construction where Quality Control (QC) referred to quality activities conducted by the contractor and Quality Assurance (QA) referred to quality activities conducted by the owner. More recently, the term Quality Assurance is being used as the umbrella term that includes the contractor's QC activities and the acceptance functions of the owner-agency. AASHTO (2006), FHWA (2008), and Code of Federal Regulations (23 CFR 637) all define the core elements of a construction Quality Assurance Program to include the following:

1. Contractor Quality Control (QC)
2. Agency Acceptance
3. Agency Independent Assurance (IA)

4. Dispute Resolution
5. Laboratory Accreditation and Qualification
6. Personnel Qualification/Certification

All six elements are deemed necessary to have a complete and effective QA Program. A QA program missing any one or more of the elements is not sufficient and should not be construed as being "substantially compliant" with the intent of the AASHTO guidelines or the federal regulation. These core elements are more fully explained in AASHTO (2006) and FHWA (2008). The contractor is responsible for performing all the QC activities to monitor, assess, and adjust their production or placement processes to ensure that the final product will meet the specified level of quality (AASHTO 2006). Acceptance is all the factors used by the Agency (i.e., certification, sampling, testing, inspection) to evaluate the degree of compliance with contract requirements and to determine the corresponding value for a given product (FHWA 2008). The acceptance activity includes monitoring of the contractor's QC activities. The other four elements ensure that appropriate processes, accredited laboratories, and qualified personnel are conducting the QC and/or QA activities. Importantly, the overall Construction Quality Assurance Program is unaffected by the use of different contract methods such as Design-Bid-Build or Design-Build.

5.4 Construction Control and Instrumentation Monitoring

Monitoring is used to provide quality assurance on methods and materials of construction and to reduce risks associated with material and property variations of the in situ soils. Often times, ground modification work is associated with construction over or through problematic ground conditions. The sampling, testing, and quantifying properties of foundation materials can be challenging; and quantifying spatial variation of properties across a site can be even more challenging. Therefore, instrumentation and monitoring during ground modification construction is routinely performed.

Field observations, including quantitative measurements obtained by field instrumentation, provide the means by which the geotechnical engineer, in spite of inherent limitations, can design a project to be safe and efficient, and the contractor can execute the work with safety and economy (Dunnicliff 1988). The primary reason to instrument and monitor the construction and performance of a geotechnical feature is to help manage risk (Marr 2013).

Historically, monitoring of geotechnical features has been performed during construction activities for verification of design (e.g., staged construction, test sections) and after construction for performance monitoring. There is much overlap between the two phases. Performance monitoring is undertaken primarily to give warning of an impending failure so that mitigation measures could be undertaken, to monitor a feature that had already moved as indicated by surface cracks, or to perform research on methods to analyze and improve the performance of the feature. Over the past twenty years or so, the reasons for performance monitoring have expanded considerably. The following are the principal reasons to monitor geotechnical performance in today's environment (Marr 2013):

1. Show that performance is safe and acceptable
2. Provide a warning of an impending failure or unsafe condition
3. Reveal unknowns and aid use of the Observational Method
4. Evaluate contractor's means and methods
5. Control construction or operations
6. Minimize damage to adjacent structures
7. Inform stakeholders
8. Satisfy regulators
9. Document performance for assessing damage
10. Improve performance and advance state-of-knowledge

Performance monitoring has long been a key tool of geotechnical engineers. During construction, activities 4 and 5 above are conducted to ensure that the contractor's work is performed according to the contract requirements, safely, and that adjacent structures are not being damaged. Further, the outcome and performance of most geotechnical features depends on the construction sequence and the contractor's means and methods. Increasingly project requirements may be in the form of a performance specification where the contractor provides the details of design, completes the work, and demonstrates, by a specifically defined requirement, the end product performs as intended. Geotechnical instrumentation can be used to determine whether the contractor's means and methods meet the specified performance requirements. The instrumentation program results, additionally, can provide sufficient data to show the potential of undesirable performance early enough to take corrective action. The instrumentation data may show why the contractor's means and methods are not working which allows them to be adjusted to reduce their impact on project performance. Hence, instrumentation can save money by helping to reduce the consequences of undesirable performance. Ineffective or inefficient aspects of the contractor's means or methods can also be identified from instrumentation data (Marr 2007a).

As Marr (2007b) indicated, today performance monitoring has exploded to provide us capabilities to measure just about anything, anywhere, and in real time. The declining cost of monitoring hardware and today's ubiquitous communications systems make real time monitoring a cost-effective construction tool. Table 1-5 summarizes the types of sensors that are frequently used in geotechnical monitoring and can be used to monitoring ground modification performance.

Table 1-5. Devices to Monitor Geotechnical Performance

Instrument	Application
Observation Well	Measure depth to ground water
Piezometer	Measure total head at a specific location
Earth Pressure Cell	Measure total normal stress in soil
Contact Pressure Cell	Measure normal stress between soil and contact with more rigid material like rock or concrete
Load Cell	Measure force in a structural member such as a strut or tieback
Settlement Plate	Measure vertical deflection at a specific point
Settlement Gage	Measure vertical deflection of one point relative to another
Deformation Monitoring Point	Measure Δx, Δy, Δz, ΔL
Flow Meter	Measure flow through a collector pipe
Flow Weir	Measure depth of flow over a weir
Crack Meter	Measure change in dimension between two points on opposite sides of a crack in 1, 2, or 3 planes
Strain Gage	Measure change in length over a known short distance
Tilt Meter, Inclinometer	Measure deviation from vertical as indicated by the pull of gravity
Borehole Extensometer	Measure change in distance between two or more points in a borehole
Geophone	Measure velocity of motion in 1 to 400 Hz range
Accelerometer	Measure acceleration of motion in 1 to 4,000 Hz range
Temperature	Measure temperature at the location of the sensor
Barometer	Measure atmospheric pressure
Automated Total Station	Measure position of multiple targets relative to fixed targets to about 0.05 inches
Global Positioning System	Measure position of one or more point relative to global reference system to about 0.05 inches
Seismographs	Measure dynamic motions resulting from shock loads such as blasting, pile driving and operation of heavy equipment

Source: Marr 2007b

5.5 Considerations of QA in Ground Modification

There is complexity in the selection and application of quality assurance methods in Ground Modification technologies. First, the contracting method for the work can affect the responsibility for the quality assurance methods. Clear communication is necessary in all contacting methods to ensure that appropriate QA methods are used and to ensure that the appropriate parties are conducting their respective responsibilities. Second, there are differences in the complexity of the QA methods used for particular ground modification technologies. The QA procedures for some technologies are simple in nature while others are more complex. Knowledge of the QA methods used in particular ground modification technologies may affect the ultimate selection or desirability of using of a technology.

6.0 COST ANALYSIS

6.1 General Cost Components

The purpose of this section is to present an overall approach that may be used to develop a cost estimate for any ground modification project. The cost estimate method described is similar to that commonly used in engineering to develop a conceptual cost estimate. That is, sufficient detail is used to identify the cost components which may have the largest effect on total project cost, however specific bid quantities and potential bidders are not necessarily known.

The method requires that a baseline cost for the particular ground modification technology be known. Over the past several decades, numerous ground modification projects have been completed in the United States and typical costs have varied during this time period primarily due to the business cycle, the experience of specialty ground modification contractors, advances in ground modification equipment, and the amount of competition from more conventional solutions. Due to the wide variety in types of ground modification technologies, there is no typical "average" price for "ground modification." Prior to describing a suggested cost estimating method for ground modification technologies, a discussion on the key factors which affect ground improvement costs is presented.

6.2 Factors That Influence Ground Modification Costs

The primary cost items for any ground modification or geo-construction technology consist of costs for mobilization, materials, labor, and the equipment for the particular technology. Added to this must be the cost of quality assurance including instrumentation and/or load tests that might be conducted as part of the QA process. There are many factors which can affect the cost of a specific project including project type, application, geoconstruction technology, soil type, labor rates, utility conflicts, location, weather, competition, etc. Identifying and understanding how these variables impact cost can be beneficial when evaluating the applicability of a ground modification or geo-construction solution.

Often the costs of the primary cost items described above are rolled into prices that are quoted in lineal feet or square feet of installed product. The role and influence of factors specific to individual technologies are discussed within the respective technical summaries and the technology cost documents within *GeoTechTools*. Typical unit costs for technologies are shown in Table 1-6.

Table 1-6. Comparative Unit Costs by Ground Modification Technology, November 2016

Category	Technology	Unit Cost
Vertical Drains and Accelerated Consolidation	PVDs, with and without fill preloading	\$0.50–\$4/lft
Lightweight Fills	Compressive Strength Fills: Geofoam; Foamed Concrete	\$75–\$150/yd^3
Lightweight Fills	Granular Fills: Wood Fiber; Blast Furnace Slag; Fly Ash; Boiler Slag; Expanded Shale, Clay and Slate; Tire Shreds	\$3–\$15/yd^3
Deep Compaction	Deep Dynamic Compaction	\$10–\$30/yd^2
Deep Compaction	Vibro-Compaction	\$5–\$9/lft
Aggregate Columns	Stone Columns and Rammed Aggregate Piers	\$15–\$60/lft
Column Supported Embankments	Column Supported Embankments	\$9/ft^2 + cost of the column
Column Supported Embankments	Columns: Non-compressible	\$30–\$80/lft
Column Supported Embankments	Columns: Compressible	\$20–\$100/lft
Soil Mixing	Deep Mixing (dry)	\$60–\$125/lft
Soil Mixing	Mass Mixing	\$15–\$75/yd^3
Grouting Technologies	Chemical Grouting	\$20/ft + \$0.65/qt
Grouting Technologies	Compaction Grouting	\$75–\$750/yd^3
Grouting Technologies	Bulk Void Filling	\$50–\$150/yd^3
Grouting Technologies	Slabjacking	\$6.50–\$9.30/ft^2
Grouting Technologies	Jet Grouting	\$250–\$750/yd^3
Grouting Technologies	Rock Fissure Grouting	\$25–\$80/ft^2
Pavement Support Stabilization Technologies	Mechanical Stabilization	\$1–\$5/yd^2
Pavement Support Stabilization Technologies	Chemical Stabilization	\$2–\$5/yd^2
Pavement Support Stabilization Technologies	Moisture Control	\$3–\$12/lft
Reinforced Soil	Reinforced Embankments	\$2–\$12/yd^2
Reinforced Soil	MSE Walls	\$30–\$65/ft^2
Reinforced Soil	Reinforced Soil Slopes	\$3–\$25/ft^2
Reinforced Soil	Soil Nailing	\$20–\$50/lft

Note that mobilization and demobilization of equipment to the project site is generally a lump sum item and can vary from a few hundred dollars to more than \$100,000 depending on the technology. The costs for the original site investigation, quality assurance testing, and instrumentation are not included in the technology unit prices or mobilization costs.

6.3 Preliminary Cost Estimation

Selection of a specific geotechnical solution should be based first on sound engineering. Routinely, two or more technologies may be identified which are potential technical solutions; when this occurs, engineers typically consider the initial cost of a solution as part of the selection process. It is important to note that while initial cost is a consideration when selecting a solution, it should not be the driving force; performance, construction time, life cycle costs, risks and safety should be factored into the evaluation of alternative geotechnical solutions.

There are many factors which can affect cost for a specific project (i.e. soil type, labor rates, utility conflicts, etc.); identifying and understanding how these variables impact cost can be beneficial when evaluating the applicability of a geotechnical solution.

To develop a preliminary cost estimate for the use of a ground modification technology information about the site conditions must be known. This information also is necessary for the preliminary design of the particular ground modification technique. This preliminary design information can then be used to develop specific quantities of materials, equipment, etc. that can used to compute a preliminary cost estimate. Cost estimating guidance for specific technologies is provided in the respective technical summaries. *GeoTechTools* (see Section 7) provides Excel spreadsheets that aid in the development of preliminary cost estimates for all of the technologies addressed within this reference manual.

7.0 GEOTECHTOOLS

GeoTechTools is a web-based geoconstruction technologies guidance and selection system. *GeoTechTools* is accessible through the website at http://www.geotechtools.org.Within this section, the development and content of the system are reviewed and summarized. The Ground Modification Reference Manual and GeoTechTools are complementary to one another. A Users Guide for *GeoTechTools* is available on the website and can be accessed from the home page.

7.1 Background, Development, Audience, and Use

GeoTechTools was developed under the auspices of the second Strategic Highway Research Program (SHRP2), which was created by the U.S. Congress in 2006 to address challenges of moving people and goods efficiently and safely on the nation's highways. SHRP2 had four main focus areas: Safety, Renewal, Reliability, and Capacity, with a number of projects under each area. Geotechnical transportation issues were addressed under the SHRP2 Renewal Focus Area, in which the goal was to develop a consistent, systematic approach to the conduct of highway renewal that is (1) rapid, (2) causes minimal disruption, and (3) produces long-lived facilities. The SHRP2 R02 project was aimed at identifying geotechnical solutions for three elements: (1) construction of new embankments and roadways over unstable soils, (2) widening and expansion of existing roadways and embankments, and (3) stabilization of the working platform. The R02 project titled: *Geotechnical Solutions for Soil Improvement, Rapid Embankment Construction, and Stabilization of the Pavement Working Platform*.

The project consisted of identification of existing and emerging geotechnical materials and systems for ground modification; the technical issues and project development/delivery methods necessary to encourage their widespread implementation; performance criteria and QA/QC procedures; and the non-geotechnical project-specific obstacles constraining full utilization of the identified systems. A catalog of materials and systems for rapid renewal projects was developed and the current state of the practice of design, QC/QA, costs, and specifications for each technology was assessed. The resulting information was cataloged in a database and made accessible through a web-based system.

The web-based information, guidance, and selection system for geoconstruction and ground modification solutions is called *GeoTechTools* and available at http://www.geotechtools.org. The value of the system is that it collects, synthesizes, integrates, and organizes a vast amount of critically important information about geotechnical solutions in a system that makes the information readily accessible to the user. The target audience for the system is primarily public agency geotechnical engineering personnel at local, state, and federal levels.

However, civil/structural, construction, pavement, and construction engineers in consulting, contracting, and academia have also found the system useful, as well as transportation managers and decision makers. Although developed for the transportation industry, the *GeoTechTools* system can be equally as valuable to other civil construction industries, and should have broad appeal to the overall geotechnical community both nationally and internationally (Schaefer et al. 2012).

The system was developed along the lines of the three elements described above; however, the final applications were divided into four areas, as shown in Figure 1-1. The system was developed with input from the research team members, the project Advisory Board, an Expert Contact Group, Federal Highway Administration, and SHRP2. Meetings were conducted throughout the project to bring together state agency transportation personnel, practitioners, contractors, and academics who work with the relevant geotechnical materials, systems, and technology areas. These meetings provided valuable information which addressed technical and non-technical obstacles limiting widespread effective use of these technologies; available best opportunities to advance the state of practice of the technologies; and future directions of these technologies in transportation work. These meetings also served to provide valuable refinements and additional information to the final system. The goal of the *GeoTechTools* system is to provide a comprehensive tool that provides guidance for applying these geoconstruction solutions to all types of transportation infrastructure.

7.2 Catalog of Technologies

The Catalog of Technologies webpage provides a listing of the ~50 geoconstruction technologies in the system. Each technology name is directly linked to a Technology Information webpage for that technology. The Technology Information page represents the technology transfer for each geoconstruction technology included in the system. Included on each Technology Information page is a series of ratings. These ratings were developed through the completion of a qualitative assessment to rate the technologies according to Degree of Technology Establishment in the U.S., Potential Contribution to Rapid Renewal of Transportation Facilities, Potential Contribution to Minimal Disruption of Traffic, and Potential Contribution to Production of Long-Lived Facilities.

From the individual Technology Information page, the user can access the following documents which are generally provided as Portable Document Format (PDF) files: Technology Fact Sheet, Photographs, Case Histories, Design Guidance, Quality Control/Quality Assurance Guidance, Cost Information, Specifications, and Bibliography. These documents were developed based on comprehensive analysis and evaluation of each technology to produce an in-depth overview that included advantages, potential disadvantages, applicable soil types, depth/height limits, groundwater conditions, material properties, project specific constraints, equipment needs, and environmental considerations. Assessments were completed for design, QC/QA, and specifications to identify key material for each technology. The downloadable documents available on the Technology Information pages result from the completion of these assessments and evaluations for each technology.

7.3 Technology Selection Guidance

The Technology Selection section was developed to aid in identifying a targeted list of potential geoconstruction technologies for a user defined set of project conditions (Douglas et al., 2012, 2014). The Technology Selection section provides both a listing of the technologies sorted by classification and a dynamic, Interactive Selection Tool. After the user identifies potential technologies, the Technology Information pages can be accessed which includes information necessary for additional screening (i.e., depth limits, applicability to different soil types, acceptable groundwater conditions, applicability to different project types, ability to deal with project-specific constraints, general advantages/disadvantages, etc.).

An experienced engineer can access solutions according to particular classifications or categories of problems. The technologies are grouped by the following classifications: Earthwork Construction, Soft Ground Drainage & Consolidation, Densification of Cohesionless Soils, Construction of Vertical Support Elements, Embankments Over Soft Soils, Lateral Earth Support, Cutoff Walls, Liquefaction Mitigation, Increased Pavement Performance, Void Filling, and Sustainability.

The Interactive Selection Tool allows the user to assess technologies based on several applications. The uniqueness of the Interactive Selection Tool is the approach of assigning a geoconstruction technology on the basis of application. The first decision in the tool is to select one of the four listed applications, which are: Construction over Unstable Soils; Construction over Stable or Stabilized Soils; Geotechnical Pavement Components including Base, Subbase, and Subgrade; and Working Platforms (shown in Figure 1-1). The Interactive Selection Tool is a knowledge based system. Special programming formed the logic and the knowledge is contained in a series of tables within the database. Each selection queries a database column and utilizes a nested if…then statement to sort the appropriate technologies. A significant benefit of the rule-based approach is the sharing of knowledge, especially when the knowledge is not the type of knowledge typically published in scholarly publications.

Similar to most geotechnical analytical solutions, the results of the analysis must be measured against the opinion of an experienced geotechnical engineer practicing in the local area of the project. The Interactive Selection Tool does not replace the project Geotechnical Engineer. The Geotechnical Engineer’s “engineering judgment” should be part of the final selection process, which takes into consideration the following project specific items:

construction cost, maintenance cost, design and quality control issues, performance and safety (pavement smoothness; hazards caused by maintenance operations; potential failures), inconvenience (a tangible factor, especially for heavily traveled roadways or long detours); environmental aspects, and aesthetic aspects (appearance of completed work with respect to its surroundings).

7.4 Products/Tools

Each technology has an individual Technology Information webpage that provides access to eight technology specific products/tools: Technology Fact Sheets, Photographs, Case Histories, Design Guidance, Quality Control/Quality Assurance (QC/QA), Cost Information, Specifications, and Bibliography.

The Technology Fact Sheet is a two-page, summary information sheet that provides basic information on the technology, including basic function, general description, geologic applicability, construction methods, transportation applications, complementary technologies, alternate technologies, potential disadvantages, example successful applications, and key references. The Photos show pictorially the equipment or methods used in the technology and can be valuable to provide a visual introduction. The Case Histories provide two-page summaries of completed projects where the technologies were used, preferably conducted in the U.S. by a State Transportation Agency (STA). These summaries provide information on project location, owner, a project summary, performance, and contact information. For some technologies relatively new to the U.S., the initial case histories were developed from projects outside of the U.S. Transportation personnel are encouraged to add Case Histories from their work to this website.

The Design Guidance summarizes the recommended design procedures for the technology. In cases where a well-established procedure (such as a U.S. Federal Highway Administration [FHWA] manual) exists, that procedure is recommended. In cases of technologies with multiple design procedures but where there is no preferred procedure, the assessment led to a recommendation of which procedure(s) to use. The Design Guidance product also identifies typical considerations which should be considered including: performance criteria/indicators, subsurface conditions, loading conditions, material characteristics, and construction techniques. The QC/QA Procedures document provides a summary of recommended monitoring practices during construction for each technology. The recommended QC/QA procedures result from an assessment of the current state of the practice of each technology. For a few technologies, design and/or QC/QA procedures were refined and improved during the system development.

For most technologies, two documents are available to assist with estimating costs. The first, a downloadable document from the Technology Information webpage titled Cost Information, provides an explanation of the cost items specific to the technology, generally emanating from the pay items contained in identified specifications. Project specific conditions and their impact on cost are discussed in the explanation. The Cost Information compiles available regional and cost numbers from STA bid tabs or national data bases when available. For technologies with limited or no STA cost history, the Cost Information provides a discussion of important considerations for the technology when estimating costs. The second document consists of a spreadsheet developed to assist in preparing preliminarily cost estimates for a technology. The spreadsheet can only be accessed through a link in the Cost Information document in order to force the user to access the cost discussion prior to developing a preliminary estimate. The Excel spreadsheet could not be prepared for some technologies due to lack of available cost information. The spreadsheet can be modified by the user to estimate specific project cost based on either a preliminary or final design. Many decisions in transportation are cost driven. In order to avoid quick elimination of technologies based solely on cost, simplified “rule of thumb” costs were avoided in the Cost Information documents. The cost spreadsheets require that a preliminary assessment or design be completed prior to estimating costs. A valid comparison of technology costs can only be completed after a preliminary design has been developed. The information system provides the user with the tools to complete a preliminary design and subsequent cost analysis which captures the technology-specific costs of implementation and construction.

A Specifications document is provided for each technology. The information presented in the Specifications document varies from identification of an existing specification which can be used for future projects, to a specification developed during system development, to a description of topics for consideration when developing a specification for a specific technology. The final document available for each technology is a Bibliography compiled during the research project. In order to assist the user in sorting the references in the Bibliography document, a reference matrix with 22 categories is provided to highlight the information in the reference, such as technology overview, design procedure, construction methods, cost, specification, QC/QA, and case history to name a few.

7.5 Summary

GeoTechTools is a web-based geoconstruction technologies guidance and selection system. Information on over 50 geoconstruction and ground modification technologies has been collected, synthesized, integrated, and organized in a single location. While *GeoTechTools* was originally developed for the transportation industry, *GeoTechTools* can be equally as valuable to other civil construction industries.

8.0 PROJECT EVALUATION AND GEOTECHNOLOGY SELECTION

8.1 Introduction

The selection of a suitable ground modification method and optimization of its design and construction to meet specific project requirements requires extensive knowledge. This knowledge must include available and technically feasible ground treatment technologies and careful evaluation of several factors. These factors include understanding the functions of the methods, utilization of several selection criteria, use of appropriate design procedures, selection and implementation of the appropriate methods for quality assurance, and consideration of all relevant cost components and environmental factors.

Evaluation and selection of an appropriate ground modification technology for a specific project should be done through a logical, sequential process. The steps in the process include a sequence of steps which proceed from simple to more detailed, allowing a best method to emerge. The process is described in the sections below.

8.2 Process to Identify Potential Poor Ground Conditions and Need for Ground Modification

The process to identify potential poor ground conditions, the need for ground modification and the selection of appropriate technologies to use follows a logical sequence. The steps involved are summarized in Table 1-7 and are explained in more detail below.

Table 1-7. Ground Modification Technology Selection Steps

Step	Description
Step 1	Identify potential poor ground conditions and need for ground modification
Step 2	Identify or establish performance requirements
Step 3	Identify and assess general site conditions
Step 4	Assessment of subsurface conditions
Step 5	Develop a short-list of geotechnologies applicable to site conditions
Step 6	Consider project constraints
Step 7	Consider project risks
Step 8	Prepare a preliminary design
Step 9	Identify alternative solutions (bridge, re-route, deep foundations, etc.)
Step 10	Evaluate project requirements, constraints, and risks against factors affecting geotechnology selection
Step 11	Compare short-list of geotechnology alternatives with geotechnology selection factors
Step 12	Select a preferred geotechnology

8.2.1 Step 1: Identify Potential Poor Ground Conditions and Need for Ground Modification

Identify project conditions which could require ground modification or geoconstruction technologies, such as projects that encounter:

- Poor ground conditions which will not provide adequate support for a transportation related structure. Poor ground conditions are typically characterized by soft or loose foundation soils, which, under load, would cause immediate or long-term deformation, or cause construction or post-construction instability.
- Project constraints which require retaining walls or steep slopes.
- Pavement foundations which require improvement.
- Construction equipment mobility issues, which require a working platform.

8.2.2 Step 2: Identify or Establish Performance Requirements

Performance requirements include deformation limits (horizontal and vertical), minimum factors of safety for stability, improved nominal resistance, stiffness parameters for pavements, drainage, and the available time for construction.

8.2.3 Step 3: Identify and Assess General Site Conditions

General site conditions consider space, constructability, and environmental constraints. Space constraints typically refer to accessibility for construction equipment to operate safely and for storage of equipment and materials onsite. Work in an urban environment might be congested to the point that construction operations and scheduling are affected. Some ground modification technologies have large equipment footprints and headroom requirements that must be considered for their use. Environmental constraints may include the disposal of spoil materials (hazardous or not hazardous), disposal of water from dewatering operations, and the effects of construction vibrations, noise, and/or dust. In urban environments these later items are of particular importance and can rule out some technologies.

8.2.4 Step 4: Assess Subsurface Conditions

The level of detail regarding the assessment of subsurface conditions will vary significantly across the wide range of transportation related projects. Regardless of the project type, the soils which will affect the performance requirements must be identified and the necessary engineering properties established to perform a preliminary design for the project. At a minimum, the type, depth, and extent must be considered, as well as the location of the ground-water table. For sites with poor ground conditions, it is further valuable to have at least a preliminary assessment of the shear strength and compressibility of the identified poor soils. For assessing the suitability of specific ground modification techniques, knowledge of loads and deformation criteria of the proposed structure can help to determine the extent of the investigation when particular techniques are under consideration.

8.2.5 Step 5: Develop a Short-List of Geotechnologies Applicable to Site Conditions

Development of a short-list, or preliminary selection, of potentially applicable technology(s) to the site conditions is generally made on a qualitative basis, taking into consideration the performance criteria, limitations imposed by subsurface conditions, schedule and environmental constraints, and the level of improvement that is required. This can be done based on experience with ground modification technologies, using this reference manual for guidance on applicable technologies, or using the Technology Selection part of *GeoTechTools*.

8.2.6 Step 6: Consider Project Constraints

Identification of constraints that might affect selection of ground modifications technologies can be done considering general and geotechnical constraints.

8.2.6.1 General

General constraints include all those items not connected with the geotechnical aspects of the project. Included are items such as the project schedule and time; budget and cost; project conditions such as right-of-way limits, geometry, scale, utilities and sequence; traffic flow/interruption; weather; environmental; availability of agency personnel, agency organization and structure, project management philosophy; and contracting processes. The contracting process, i.e., project delivery method, and the role it might have on use of these techniques for the specific project should be assessed. Identification of as many of these constraints affecting the project as possible will allow better selection of appropriate ground modification technologies.

8.2.6.2 Geotechnical

The geotechnical constraints considered here are those specific to geotechnical aspects of the project and ground modification technologies that might be used. Such constraints include subsurface conditions, including archaeological remains, contaminated groundwater, obstructions, and buried utilities; proprietary products/processes; knowledge and experience with specific technologies; allowable movements-both vertical and horizontal; stability requirements; noise and vibrations due to construction equipment; time for improvement to be effective; spoils development and disposal; environmental impacts of the technology;

availability of qualified contractors, personnel, materials and specialty equipment; and constructability of the selected technology. Identification of the poor soil types, their depths and extent, along with their engineering properties, is perhaps the most important consideration here in terms of selecting appropriate ground modification technologies.

8.2.7 Step 7: Consider Project Risks

Project risks can occur from not meeting schedule and budget targets, from not identifying geotechnical issues correctly, and from not providing an appropriate or adequate fix to the poor ground situation. Schedule and budget risks can occur due to contractual issues between the general contractor and a specialty contractor, a late notice to proceed, weather affecting production, site access issues, material supply problems, equipment availability issues, and so on. These risks need to be continuously identified and addressed as the project proceeds. Identification of geotechnical risks generally follows from consideration of the possible occurrence of such issues as slope instability, settlement, liquefaction, contamination, problem soils, etc. These should be identified as part of the subsurface investigation and carried forward as potential risks that can affect the project schedule and performance. Selection of a ground modification method for the project conditions that turns out not to work on the site soil conditions is also a risk that should be identified.

8.2.8 Step 8: Prepare Preliminary Designs

Based on collection, synthesis and consideration of all the above items, a preliminary list of suitable and appropriate ground modification technologies for the project can be developed. To compare the various improvement technologies, preliminary design of each candidate method for the particular site conditions must be completed. The design for each technology will be unique to fit the project performance criteria and constraints. Upon completion of preliminary designs for the candidate technologies, preliminary cost estimates can be developed and compared.

8.2.9 Step 9: Identify Alternative Solutions (Bridge, Re-route, Deep Foundations, etc.)

There always exist alternate solutions available, and these should be evaluated along with the ground modification methods. Alternative solutions include the use of a bridge system, the use of deep foundations to support the structure rather than ground modification, or perhaps rerouting the alignment to avoid the poor ground conditions.

8.2.10 Step 10: Evaluate Project Requirements, Constraints, and Risks Against Factors Affecting Geotechnology Selection

For this step, additional geotechnology selection factors are considered. These factors are in addition to the primary factors of application, purpose of ground modification, soils type(s), depth of soils to be improved, etc. considered in the Step 5 development of a short-list of applicable technologies. Additional selection factors may vary by application and project, but may include the following factors: constructability (i.e., availability of material, equipment, staging, access, groundwater, and etc.); speed of construction; construction disruption (amount to traveling public and/or adjacent structures); longevity of constructed works (service life and maintenance requirements); ROW requirements or restrictions; aesthetics; environmental aspects and concerns; local familiarity with geotechnology; degree of establishment of the geotechnology; design procedure (availability, complexity, codified, etc.); contracting means and practices; cost (initial and life cycle); additional project constraints; and project risks. In this step constraints and risks are explicitly evaluated for their effect on the project.

This listing is not project specific and is presented as a tool to help evaluate the short-listed geotechnologies developed in Step 5. For a given project, additional selection factors may be applicable. Many of items may be of little to no importance on a particular project.

The evaluation should be performed by the party that is responsible selection of the geotechnology to be used. Depending on the contracting method used for the project, this evaluation may be performed by the owner, the consulting engineer representing the owner, or the contractor.

Each geotechnology selection factor is evaluated based on its relevancy and importance to the project requirements and site constraints and they are assigned a rating number between zero (0) and three (3). This is termed the weight factor (WF) for a given geotechnology selection factor. Three is assigned to the most relevant or important factors, one is assigned to the least relevant ones and zero (or NA-not applicable) is assigned to non-relevant items. The evaluation results should be tabulated as shown in Table 1-8. For example, speed of construction, amount of disturbance, longevity of constructed works, and cost are typically the most important wall selection factors for permanent wall systems.

Table 1-8. Geotechnology Selection Factors

Geotechnology Selection Factor	Weight Factor (WF)
Speed of construction	0 through 3
Minimize construction disturbance	0 through 3
Longevity of constructed works	0 through 3
Cost of construction	0 through 3
Constructability	0 through 3
ROW requirements or restrictions	0 through 3
Aesthetics	0 through 3
Environmental	0 through 3
Degree of establishment	0 through 3
Familiarity with geotechnology	0 through 3
Design procedure	0 through 3
Contracting	0 through 3
Life-cycle cost	0 through 3
Additional project constraint	0 through 3
Additional project constraint	0 through 3
Project risk	0 through 3
Project risk	0 through 3
Project risk	0 through 3

8.2.11 Step 11: Compare Short-List of Geotechnology Alternatives with Geotechnology Selection Factors

The applicability of all the short-listed alternatives (from Step 5) should be evaluated against Geotechnology Selection Factors that are defined here using a rating between one (1) and four (4). This is a relative rating (RR) of how well the short-listed technologies meet the specific geotechnology selection factor for this project. For each factor, four is assigned to the most suitable factor for the geotechnology being evaluated and one is assigned to the least suitable factor for the geotechnology being evaluated. These ratings are selected based on experience with the short-listed geotechnologies in meeting the selection factors.

8.2.12 Step 12: Select a Preferred Geotechnology

This is the final step where alternatives are compared to each other in a geotechnology selection matrix format and the one that has the highest score is selected for the project. The scoring of each alternative is obtained for each geotechnology selection factor by multiplying WF from Table 1-7 with the 1 through 4 relative rating for each to obtain the weighted rating (WR) for each geotechnology selection factor. The WR for all the selection factors are then added together to obtain the Total Score for each short-listed technology. The geotechnology which has the highest score should be developed as the base design. Other high scoring geotechnologies may also be included in the Contract Documents as acceptable alternates. An example project selection matrix is shown below in Section 8.4.

8.3 Additional Considerations – Detailed Subsurface Investigation, Design, and Cost Estimate

Once a geotechnology is selected additional information may be necessary to develop the final design and project cost estimate. Ground modification technologies often require specialized geomaterial parameters for design and construction that are not obtained during preliminary site investigations. Supplemental detailed subsurface investigations are sometimes necessary to obtain the additional information. Details of needed geomaterial parameters are provided in the technology summaries.

8.4 Geotechnology Selection Example

The Geotechnology Selection Factors are factors on which the project selection will be based and are different for each project. An example of a project selection matrix is shown in Table 1-9.

Table 1-9. Sample Project Selection Matrix

Geotechnology Selection Factor	Weight Factor	Geotech A Relative Rating	Geotech A Weighted Rating	Geotech B Relative Rating	Geotech B Weighted Rating	Geotech C Relative Rating	Geotech C Weighted Rating
Speed of construction	2	3	6	4	8	2	4
Minimize construction disturbance	2	1	2	2	4	2	4
Longevity of constructed works	3	4	12	4	12	4	12
Cost of construction	2	3	6	2	4	1	2
Constructability	3	2	3	2	3	2	6
ROW requirements or restrictions	NA	0	0	0	0	0	0
Aesthetics	NA	0	0	0	0	0	0
Environmental	1	0	0	0	0	1	1
Degree of establishment	3	3	9	2	6	2	6
Familiarity with geotechnology	2	2	4	1	2	2	4
Design procedure	2	3	6	2	4	2	4
Contracting	1	4	4	4	4	3	3
Life-cycle cost	1	3	3	3	3	3	3
Constraint – construction season	3	2	6	2	6	1	3
Risk – delay due to settlement time	3	4	12	2	6	2	6
Risk – quality assurance	2	1	2	3	6	3	6
TOTAL SCORE			**75**		**69**		**64**

Example factors are shown in the table but others can be added as necessary. The Geotechnology Selection Factor Weight Factors are determined in Step 10 and Table 1-8. The Geotechnology Relative Ratings are determined in Step 11. The Total Score for each geotechnology is the sum of the Weighted Ratings as summarized at the bottom of the table. In this example, Geotechnology A would be advanced to final design and cost analysis, and Geotechnology B may be accepted as an alternate.

8.5 Combination of Geotechnologies

Combinations of technologies should be considered in the geotechnology selection process. Typical combinations are noted within the Complementary Technology Sections, within each technology summary. Complementary technologies are also listed on the Technology Fact tool on the http://www.geotechtools.org website.

There are many possible combinations of technologies, as listed in Table 1-10.

However, there are a number of combinations that are more likely to be successful. For example, blast densification does not densify the soil in the upper 5 feet of soil below the ground surface. This soil can be compacted by a shallow compaction method such as rapid impact compaction. Prefabricated vertical drains and fill preloading are often used together to accelerate consolidation and the construction schedule. Lightweight fill material is used to reduce the load on column supported embankments. In some cases, the subsurface conditions across a site or project vary significantly enough to warrant the use of different ground modification technologies. Numerous successful combinations that were found during a literature review on embankments over unstable ground (Barngrover et al. 2013) are noted in Table 1-10.

Table 1-10. Technology Combinations Found in Literature Review

Technology	PVDs and Fill Preloading	Vacuum Preloading with and without PVDs	Vibro-Compaction	Deep Dynamic Compaction	Blast Densification	Aggregate Columns	Geotextile Encased Columns	Deep Mixing Methods	Jet Grouting	Soil Nailing	Mechanically Stabilized Earth Walls	Reinforced Soil Slopes	Lightweight Fills	Column Supported Embankments	Micropiles
PVDs and Fill Preloading			X			X		X			X		X		
Vacuum Preloading with and without PVDs							X								
Vibro-Compaction	X				X	X									
Deep Dynamic Compaction					X	X					X				
Blast Densification			X	X											
Aggregate Columns	X		X	X							X				
Geotextile Encased Columns		X										X			
Deep Mixing Methods	X										X		X		
Jet Grouting										X					X
Soil Nailing									X						X
Mechanically Stabilized Earth Walls	X			X		X		X							
Reinforced Soil Slopes							X								
Lightweight Fills	X							X						X	
Column Supported Embankments													X		
Micropiles									X	X					

Source: Barngrover et al. 2013
Blank cells=No case histories located for that combination

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Chapter 2

VERTICAL DRAINS AND ACCELERATED CONSOLIDATION

CONTENTS

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1.0 DESCRIPTION AND HISTORY

1.1 Description

Consolidation of soft ground using vertical drains and preloading is a technique used since the 1920s. The vertical drains provide a shortened pathway for water to exit the soils while fill preloading surcharges the foundation soils. Initially sand drains were used, then cardboard drains, followed by geotextile-encased drains (prefabricated vertical drains). The most common vertical drain at present is the use of prefabricated vertical drains (PVDs). PVDs are often called *wick* drains in the United States, but the term PVD is more appropriate since drainage is via pressure, and not by wicking. PVDs are used to accelerate consolidation of soft saturated compressible soils under load.

PVDs are band shaped (rectangular cross-section) products consisting of a geotextile jacket surrounding a plastic core with drainage channels. This configuration permits pore water in the soil to seep into the drain for collection and transmittal up and down the length of the core. While there are some variations, the size of a PVD is typically 4 inches wide by ⅛ to ⅜ inches thick. Basic evolution of the term *wick* drains comes from the idea that they look like a wick, and the first available product was known as a "cardboard wick." Other common terms for these drains are drainage wicks, band drains, and strip drains.

PVDs are only one general type of vertical drain system. Aggregate columns (see Chapter 5) and geotextile encased columns (see http://www.GeoTechTools.org) also provide vertical drainage. The general principles that govern all vertical drain installations are similar for all types of drains. However, the cost advantages of PVDs over other vertical drain systems have resulted in their almost exclusive use except in unusual circumstances as outlined under Section 2.4 Limitations.

The most common transportation use of PVDs is to accelerate consolidation for approach embankments at bridges or other embankment construction over soft soils, where the total post construction settlement would otherwise be unacceptable. A typical cross-section of a PVD installation for embankments is shown in Figure 2-1.

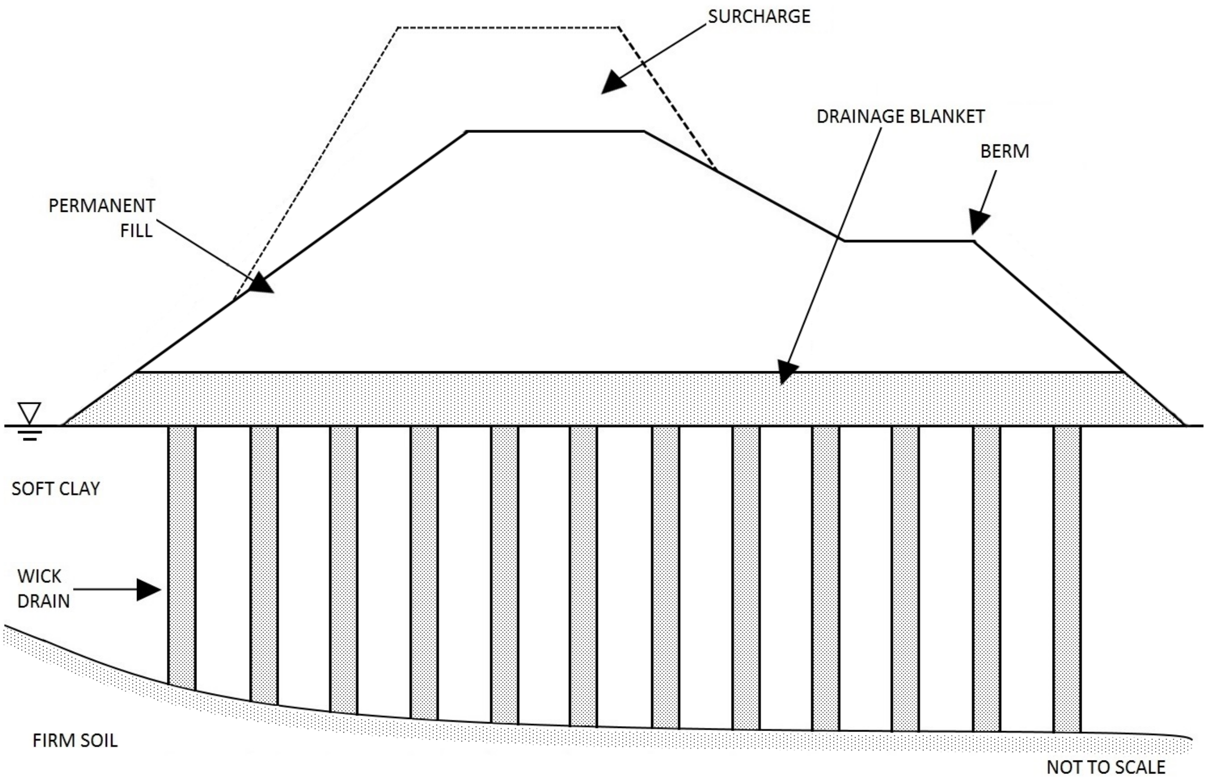

Figure 2-1. Prefabricated vertical drain installation for a highway embankment.

A preload and a surcharge are shown in Figure 2-1. Preload is defined as the application of load to induce settlement and consolidation of the foundation soils. Often this is the amount of fill material necessary to bring the site to the final elevation, including the amount of additional embankment necessary to accommodate the final amount of settlement. Surcharge is the extra load or the added fill above final elevations that is used to accelerate settlement and/or minimize secondary consolidation; it is removed after the desired consolidation is achieved.

Consolidation of compressible soils by vacuum preloading was conceptually introduced in the 1950s and has recently evolved as a reasonably reliable technology. The basic premise for vacuum consolidation consists of removing atmospheric pressure from a confined and sealed soil to be consolidated and maintaining the vacuum for a predetermined period of time. When used in combination with PVDs, the soil is loaded approximately uniformly throughout its depth by the equivalent of one atmosphere of vacuum. Although used extensively outside the United States, vacuum preloading has seen little use in the United States and will not be discussed further herein. Information on vacuum preloading can be found on http://www.GeoTechTools.org under the Vacuum Preloading With and Without PVDs technology and the references therein.

1.2 Historical Overview

The development of PVDs almost parallels the development of the vertical sand drain concept. A United States patent for a sand drain system was granted in 1926 to D. J. Moran. The California Division of Highways, Materials and Research Department, conducted laboratory and field tests on vertical sand drain performance as early as 1933. From that time until the early 1980s, sand drains were used almost exclusively for highway projects across the United States that required a vertical drain solution.

In Europe, the lack of available sand with suitable drainage characteristics and the presence of certain environmental drawbacks led to the development of the first PVD in the late 1930s by Walter Kjellman, then Director of The Swedish Geotechnical Institute. This drain utilized three layers of cardboard, with the two outer layers serving as a filter and the middle layer as a 10-channel separator. The drain measured 4 inches wide by ⅛ inches thick and became known as the "Cardboard Wick." This drain was used extensively in Sweden after 1939, when a machine for installation was developed. Later these drains found their way into other countries. Sweden subsequently, however, reverted to the use of sand drains, mainly because of the low permeability, poor resistance to pressure, and the low water transportation capability of the cardboard material. In spite of this, Oleg Wager, a colleague of Kjellman at The Swedish Geotechnical Institute, believed so strongly in the potential of the PVD that he worked on and developed an improved drain, and in 1972 patented the Geodrain. This drain had essentially the same dimensions as the "Cardboard Wick" but was constructed differently. The Geodrain consisted of a plastic core containing 27 grooved channels for water transportation, and was surrounded by a filter material made of paper. PVDs, as we know them today, had finally arrived. Almost simultaneously the Castleboard Drain was developed in Japan. It was very similar in appearance to the Geodrain, but with a geotextile material, instead of paper, used as a filter.

PVDs did not come into use in the United States until the mid-to-late 1970s. Prior to this, the use of jetted and augered nondisplacement sand drains had proved to be an effective vertical drain system. Another contributing factor to this timing was that all of the practical PVD experience had been attained mostly in Europe and Japan.

With environmental and economic factors slowly eliminating the use of jetted and augured sand drains, a newly designed PVD material known as Alidrain® was introduced in the United States. Acceptance, while initially slow due to the lack of prior experience and design procedures, grew quite rapidly. Today there are hundreds of projects completed yearly. In fact, PVDs are now used almost exclusively where a vertical drain solution is required.

Since the development of the first cardboard wick, there have been over 50 types of PVDs used worldwide and at least 10 in the United States. Historically, most of those installed in the United States are one of two basic designs. Both have an independent outside geotextile filter, but the core of one type is corrugated, and the other is studded. The corrugated design results in straight channels for flow. This type of PVD design currently accounts for nearly 100 percent of the United States market. Commonly used brand names include AMERDRAIN 407®, MEBRA-DRAIN 7407®, MEBRA-DRAIN MD 88®, COLBONDDRAIN CX-1000® and ALIWICK®. A typical PVD of this type is shown in Figure 2-2 (top).

The studded core-type design results in a more turbulent flow pattern, thereby necessitating a slightly thicker drain in order to obtain the same flow rates. Common studded core brand names are ALIDRAIN® and AMERDRAIN 417®. These devices account for a small percentage of the United States market because of their slightly higher cost. A typical PVD of this type is shown in Figure 2-2 (middle).

There are no available data from completed projects that would indicate any advantages or disadvantages for the two types of cores. Much of the early usage was with the studded design, but currently the grooved design is used most often, based on its lower material cost.

Recently, high capacity vertical composite drains have been introduced to mitigate liquefaction (Rollins et al. 2004 and Rollins and Strand 2007). Termed earthquake drains (EQ drains), these drains consist of 3 to 6 inch diameter corrugated, perforated drain pipe wrapped with a geotextile filter fabric. A typical EQ drain is shown in Figure 2-2 (bottom).

MebraDrain MD4707® (top left), Ameridrain 407® (top right), TenCate Alidrain® (middle), Nilex Construction®, after Utah DOT (bottom)

Figure 2-2. Typical prefabricated vertical drain products: PVDs with corrugated inner core (top), PVD with studded inner core (middle), vertical earthquake drain (bottom).

1.3 Focus and Scope

The principal purpose of this chapter is to acquaint the reader with the use of vertical drain methods. Many of the concepts and ideas stated herein are taken from practical experience and the basic references, listed in Section 1.5 Primary References and in the references at the end of this chapter.

The content of this technical summary includes the following:

- Applications
- Feasibility
- Construction procedures and monitoring
- Design concepts
- Specifications
- Case histories

The major use of vertical drains is for consolidation of soft soils by preloading and/or surcharging. Other PVD applications include: pressure relief wells to reduce pore pressures due to seepage; lowering perched water table conditions; and reducing liquefaction potential in soils.

When used in conjunction with surcharging or preloading, PVDs have the following principal benefits:

- To decrease the settlement time required such that final construction can be completed in a reasonable time, with minimal post-construction settlement
- To decrease the amount of surcharge or preload material required to achieve a settlement in the given time
- To increase the rate of strength gain due to consolidation of soft soils when stability is of concern

Any one of these benefits may be the sole reason for use on a particular project, or any combination of benefits may be the desired result.

1.4 Glossary

Consolidation – a time-dependent settlement process that occurs in saturated fine-grained soil that have a low coefficient of permeability.

Earthquake drains (EQ drains) – high flow capacity synthetic vertical drains installed with a vibrating mandrel consisting of a corrugated plastic pipe three to six inches in diameter wrapped with a geotextile filter.

Geosynthetic – a planar product manufactured from a polymeric material used with soil, rock, earth, or other geotechnical–related material as an integral part of a civil engineering project, structure, or system.

Geotextile (GT) – a permeable geosynthetic made of textile materials, used as a separator between base, subbase and subgrade layers, used as filters in drainage features, and used in stabilization of soft subgrade layers.

Preconsolidation stress – the maximum past effective overburden stress to which a soil has been subjected.

Prefabricated vertical drain (PVD or PV drains) – band shaped (rectangular cross-section) products consisting of a geotextile jacket surrounding a plastic core with drainage channels. Water flows from soil through the filter into the core of the drain and from there upward and/or downward to the nearest free draining layer.

Preloading – application of a load to site soils to induce settlement or consolidation of the foundation soils. The load may be part of the permanent embankment or be temporarily applied.

Primary consolidation – The compression of the soil under load that occurs while excess pore pressures dissipate with time.

Secondary compression – time dependent settlement occurring at constant effective stress with no subsequent changes in pore water pressure.

Surcharge – an extra load applied above the preload to accelerate consolidation or minimize secondary compression.

Wick drain – another name for prefabricated vertical drain (PVD).

1.5 Primary References

The primary references for Vertical Drainage and Accelerated Consolidation are the following:

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2.0 FEASIBILITY CONSIDERATIONS

2.1 Applications

2.1.1 Preloading of Soft Soils

In practice, PVDs are most commonly used in consolidation situations where the soil to be treated is a moderately to highly compressible soil with low permeability and is fully saturated in its natural state. Such soils are typically described as silts, clays, organic silts, organic clays, muck, peat, swamps, muskeg, or sludge.

In general, soils for which PVDs are being used must be saturated and normally to slightly over-consolidated prior to loading. In addition, PVDs are used in under-consolidated soils in land reclamation applications. The loading should exceed the preconsolidation stress (the maximum past effective overburden stress) of the soils for the PVDs to be totally effective.

Ordinarily, PVDs are not used in highly organic materials, or where secondary consolidation will result in significant post-construction settlement. However, additional surcharging may be used with this solution to minimize the effect of secondary consolidation.

Many of the common uses of PVDs are illustrated in Table 2-1, but are not to be considered all-inclusive.

Table 2-1. Common Uses of PVDs for Transportation Applications

Field of Application	Purpose is to Increase Stability	Purpose is to Accelerate Settlements
Highways Roadway Embankments	Yes	Yes
Highway Structure Approach Fills	Yes	Yes
Airfield Runways and Taxiways	Yes	Yes
Earth Embankment Dams	Yes	Yes
Storage Tanks	Yes	Yes
Pile Foundations to Reduce "Negative" Skin Friction	No	Yes
Liquefaction Mitigation	Yes	Yes
Land Reclamation	Yes	Yes

2.1.2 Special Application, Liquefaction Reduction

One of the potential uses of PVDs that is currently emerging, is the installation of drains to reduce the potential for liquefaction of saturated granular soils. The PVDs provide a pathway for excess pore water pressures developed by seismic excitation to dissipate rapidly through the drainage pathway provided by the PVDs. In this application, the PVDs simply replace the gravel drains that have been in use for liquefaction mitigation for the past 30 years. The PVDs for these applications are the EQ Drains, which are vertical, slotted plastic drain pipes of 3 to 6 inches in diameter and have a high flow capacity. The high flow capacity provides rapid dissipation of excess pore water pressures that, if not dissipated, would lead to liquefaction of the soils.

Limited research has been conducted to determine the extent of potential liquefaction reduction due to EQ Drains. However, it appears that some degree of improvement will be obtained. Additional information can be found in Rollins et al. (2004) and Rollins and Strand (2007).

2.2 Advantages and Potential Disadvantages

2.2.1 Advantages

2.2.1.1 Economy

For typical projects, the cost for PVDs is much less than available alternatives such as aggregate drains.

2.2.1.2 Installation

Generally, the production rate for PVDs will average between 15,000 to 20,000 lineal feet per day per rig. Production rates as high as two to three times these figures have been achieved, but for planning purposes, 15,000 feet per day is generally used.

2.2.1.3 Continuity of Drain

PVDs provide an assurance of a permanent drainage path, even with considerable lateral displacement or buckling under vertical or horizontal soil movements.

2.2.1.4 Minimal Displacement

The typical size of mandrel used for PVDs is small enough to create minimal displacement during installation. The displacement effects on permeability of the disturbed soil can be taken into account in the PVD design process that leads to an increase of time needed for consolidation. There has been no evidence of a significant reduction of shear strength of in situ soils due to remolding.

2.2.1.5 Improved Quality Control

The quality control and assurance for PVD construction is quite simple. Because continuity of the drain is assured during installation, the major duties of inspection are to ensure proper drain anchorage and proper depth attainment.

2.2.1.6 Equipment Flexibility

There are many types and sizes of PVD installation equipment that can be easily adapted to field conditions. PVDs are generally installed with a static or vibratory installation force, and normal equipment can be adapted for a minimal amount of jetting, where necessary. Very lightweight equipment can also be used in unstable ground conditions.

2.2.1.7 Low Material Storage

PVDs come in reels usually containing 450 to 1,000 feet of material. Each roll is approximately 3.3 feet in diameter, 4 inches thick, and can be easily stored.

2.2.1.8 No Spoil Removal

With the exception of situations where pre-augering or drilling through stiff layers is necessary to reach the soft compressible soil layer, there is no significant excess spoil material to be removed from the site.

2.2.1.9 No Water Required

Except in unusual cases, PVDs are installed without jetting. Even if a minimum amount of jetting is required, the resulting surface runoff is minimal.

2.2.2 Potential Disadvantages

2.2.2.1 Headroom Limitations

PVD installation equipment must be 5 to10 feet taller than the depth of installation which can limit their use for some sites.

2.2.2.2 Materials Must be Stored Properly

PVD material can degrade in sunlight and, therefore, must be stored properly. While most specifications require the material to be covered during storage, the effect of sunlight on the geotextile will not be significant unless the material is on site for more than a month.

2.3 Feasibility Evaluations

When PVDs are used to accelerate settlement during loading, the subsoil must meet the following criteria:

- Moderate to high compressibility
- Low permeability
- Full saturation
- Final embankment loads must exceed preconsolidation stress
- Secondary consolidation must not be a major concern
- Low-to-moderate shear strength

Providing the soils meet the above geotechnical criteria, the project still must be evaluated for the possible effects of environmental and other site conditions.

2.3.1 Geotechnical Criteria

To ascertain the geotechnical criteria to ensure a successful design, the subsurface investigation must be sufficiently extensive to determine the extent and depth of the compressible soils and to secure high-quality undisturbed samples for consolidation and strength testing. In the preliminary phase, laboratory testing consisting of Atterberg Limits, moisture content and organic content within short depth intervals is extremely useful in determining strata changes and providing insight as to the depth in which undisturbed samples should be recovered. Sufficient consolidation testing must be performed to determine the over-consolidation ratios (OCR) with depth as well as shear strength testing to determine both undrained and drained shear strengths. The use of piezocone (CPT) soundings is valuable for identifying thin interbedded granular layers, which affect settlement-time extrapolations. Piezocone pore pressure dissipation tests could be used to determine field coefficients of consolidation, although the literature indicates that c_h obtained from piezocone methods typically varies by half an order of magnitude from actual values (Bartlett et al. 2001).

2.3.2 Environmental Considerations

If the in situ soils are contaminated with any kind of hazardous waste or material, then it is possible that the excess pore water draining through the PVD will need to be collected and treated. In such situations, care must be exercised to prevent the PVD from penetrating into a highly permeable layer, should one exist below the compressible stratum.

2.3.3 Site Conditions

Site topography and in situ soil conditions can have a considerable effect on the economics of a PVD solution. Some of the specific site and soil conditions that affect the economics or feasibility are listed below and subsequently discussed:

- Uneven working surface
- Limited headroom
- Obstructions above the compressible layer
- Unstable working surface
- Depth of PVDs in excess of 100 feet
- Stiff to very stiff compressible layers
- Extremely soft layer for anchoring
- Poor site accessibility
- Overhead or subsurface utility interference

Uneven working surface. PVDs cannot be installed economically on steep slopes. Therefore, the area will have to be benched with widths sufficient to allow for the equipment. Generally, a minimum bench width of 25 feet is required.

For shallow depths, PVDs can be installed on slopes as steep as one on five. Deeper drains will require a level working surface to ensure uniform drain spacing at all depths. A constant minor slope is preferable to an undulating surface, and it also facilitates the construction of a more effective drainage blanket.

Limited headroom. A rule of thumb is that the depth of a PVD needs to be 10 feet shorter than the available headroom in order to be economically installed. Limited headroom situations occur most often when installing under an existing bridge. PVDs can be installed vertically in segments with limited headroom, but the cost would most likely be as high as five times the normal unit installation price.

Obstructions above the compressible layer. Where obstructions must be penetrated above the compressible layer, considerable extra costs could be involved. A stiff or dense upper layer that can be penetrated without pre-drilling will add minimally to the cost. Obstructions such as concrete, rock, rubble, slag, brick, wood, riprap, stone, debris, rubbish, or trash usually require pre-drilling. Dense layers with SPT blow counts greater than 10 to 15 may require pre-drilling.

Unstable working surface. In general, most unstable working surfaces can be made stable prior to the installation of PVDs with the use of geosynthetics and granular soil for the drainage layer (2 to 3 feet thick). The installation equipment will usually penetrate these materials without difficulty. Where the ground cannot be stabilized, movable timber mats may be used for support, or specialized lightweight equipment is available at a substantial increase in the unit cost.

Extreme depth. PVDs have been installed to depths of 200 feet with the use of specialized equipment. A rule of thumb is that PVDs over 100 feet in depth will require a crane for installation. Depths over 120 feet require a very large crane and specialized installation masts.

Stiff-to-very-stiff compressible layers. If the layer that is considered compressible is quite stiff, the entire length of PVDs may need to be pre-augered or predrilled. In such cases, it is not normally advisable to use PVDs. The pre-augering or drilling will create a large void area around the drain and the subsequent collapse will result in excessive soil disturbance. If the void could be filled with sand, a PVD is not necessary to begin with.

Extremely soft layer for anchoring. In some cases, designers have selected a depth that does not fully penetrate the compressible layer. In such soil with a very low shear strength, it may become very difficult to anchor the drain at that depth, and either additional depth will be necessary or special equipment procedures will be required. This situation slows production and adversely impacts cost.

Site accessibility. While the equipment for PVD installation is relatively easy to transport, there are some situations where site accessibility may add to the cost. These include multiple overhead obstructions on a single project or steep access roads or site grades. In other cases, costly access roads may be necessary to transport the equipment down steep slopes or across unstable areas.

Overhead or subsurface utility obstructions. Usually underground utilities can be located prior to PVD installation, and drains can be installed around them to avoid any problems. However, large sewer pipes intermixed with several other utilities may create a situation where drains cannot be installed for a significant width. Overhead wires can possibly present more of a logistic problem. If the wires cannot be de-energized, significant widths of treatment might need to be eliminated, or the use of angled drains specified.

Should any of the above site conditions be encountered, it would be advisable to contact specialty contractors experienced in the installation of PVDs in order to determine the magnitude of difficulty. All of the above cases have been encountered and overcome to install drains successfully. However, the additional costs can be very significant.

2.4 Limitations

It is important to remember that a PVD serves no structural function (except perhaps in liquefaction reduction). By providing a shorter drainage path, PVDs provide a faster release of excess pore water, thereby resulting in faster settlement and quicker strength gain by consolidation. For sites with a stability problem, the soil will initially have the same strength with or without PVDs installed. Therefore, in situations where stability is of concern, the rate of increase of load must be carefully controlled (staged construction), monitored, or if the final or stage fill height is less than 12 feet, vacuum consolidation may be considered.

PVDs used without a surcharge load can accelerate only primary consolidation, i.e., water being squeezed out of the soil. Therefore, it is important to estimate the magnitude and time rate of secondary consolidation. Secondary consolidation is caused by the soil particles reorienting or deforming under constant load, and is not dependent on water being squeezed out of the soil (FHWA 2006). Secondary settlement can be minimized by placement of excess surcharge and/or extension of waiting periods prior to final construction. Soils with high organic content are prone to significant secondary consolidation and should be carefully evaluated for the potential magnitude.

Other limitations on the use of PVDs should be considered. PVDs have been installed at depths of up to 200 feet. However, 100 to 120 feet is a typical depth limit with specialized equipment required for depths greater than 120 feet.

In most situations, the flow properties of a good quality PVD will not inhibit consolidation times. However, for extremely deep PVDs, combined with heavy loading and relatively high in situ soil permeability, the flow capacity of the system could be a limiting design consideration. In these rare cases, well resistance in the drain occurs, and consolidation time will be determined more by discharge capacity of the drain than by the horizontal permeability of the in situ soils. Rixner et al. (1986a) and Koerner (2012) contain specific technical guidance for this condition.

PVDs are not recommended where the entire length or lower length requires pre-drilling. In these cases, sand drains or other methods of ground modification may be considered.

2.5 Alternate Solutions

Alternate solutions can be functionally divided into four categories, as follows:

1. Accept time constraints without the use of any vertical drain system.
2. By-pass the compressible soil, using deep foundation elements.
3. Reduce the compressibility of the in situ soil, using a different ground modification method.
4. Reduce the loading on the compressible soil, using lightweight fill.

When there is sufficient time for settlement to occur under the final load conditions, PVDs obviously are not needed. In some cases, a preload without the use of any PVDs may be all that is necessary to obtain consolidation within the allowable time constraints. The cost of this preload should be compared against the cost of using PVDs with preload.

An alternate method would be to design for excessive post-construction settlement and accept the anticipated cost of repairs or corrections to the ground or structure as a long-term maintenance responsibility.

Use of a deep foundation is an effective and expensive means of bypassing compressible soils. Deep foundations may be used to support a bridge or an embankment, using a column supported embankment (see Chapter 6). Such solutions are usually much more expensive and may also limit the flexibility of future uses of the site.

Reducing the compressibility of in situ soils offers the greatest variety of solutions. While usually more expensive than PVDs, the following are alternative methods of improving the soil:

- *Stone columns.* This method is used in soft subsurface soils to accelerate settlement, reduce magnitude of settlement, and provide sufficient strength increase to preclude deep-seated global failure. A variant using geotextile encased columns (GEC) may also be considered for very soft soil profiles. (See Chapter 5).
- *Deep soil mixing.* This method is used to change the in situ compression and strength characteristics of soils. (See Chapter 7).
- *Excavation and Replacement*. This is a method of removing the compressible soils and replacing with a compacted, good quality fill material.
- *Lightweight fills.* This method is used to reduce the embankment load. (See Chapter 3).

Stone columns and deep soil mixing will strengthen the soil and reduce its compressibility. Where stability is a significant problem, combined solutions may be warranted in order to achieve the desired result. For example, the unstable areas can be treated with stone columns or deep soil mixing, and the remaining areas treated with PVDs, or excavation of weak in situ soils and replacement with granular materials.

The use of lightweight fills to minimize the total amount of settlement has been used more frequently in recent years. While lightweight fills do not result in soil improvement, they reduce settlement and stability problems by reducing imposed loads.

3.0 CONSTRUCTION AND MATERIALS

3.1 Construction

3.1.1 Prefabricated Vertical Drains

The various methods of prefabricated vertical drain installation, which employ mostly the same principles, are listed in Table 2-2.

Table 2-2. Methods of Prefabricated Vertical Drain Installation

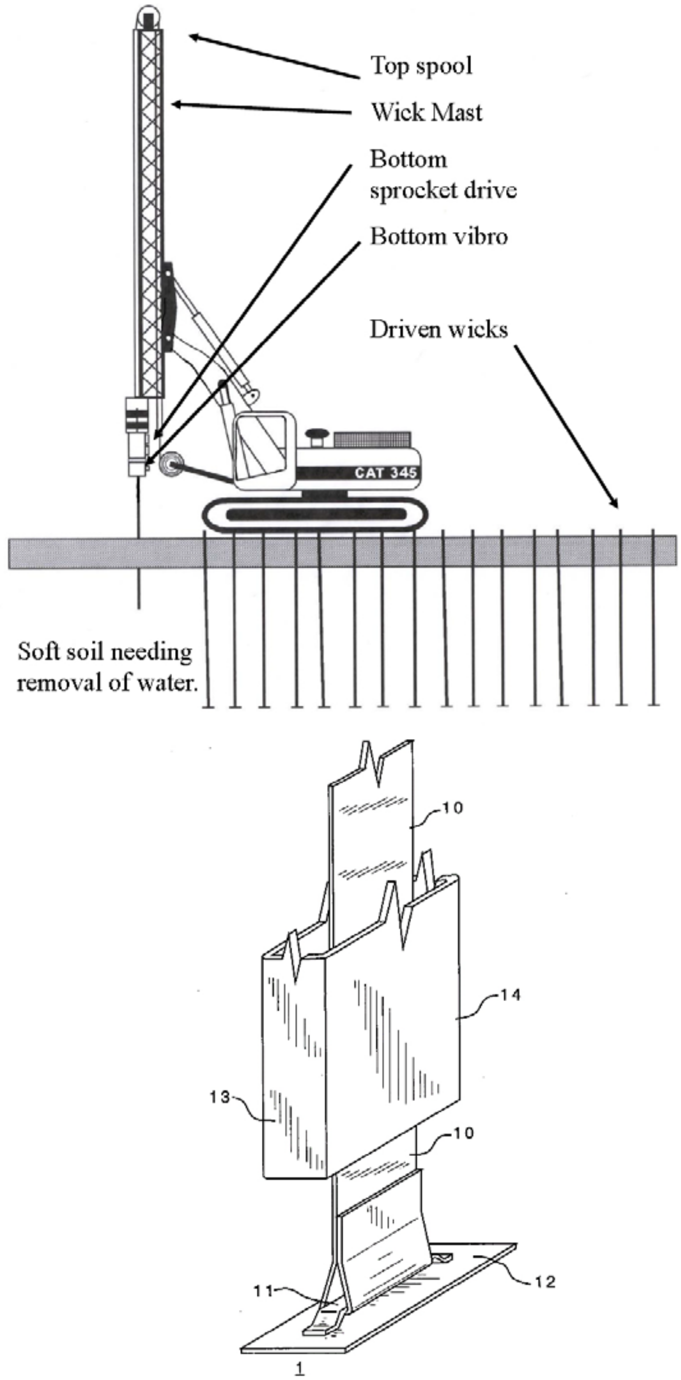
Installation Method	Method Options
Static	1. Chain Driven* 2. Sprocket Driven* 3. Cable Pull-down* 4. Heavy Weight 5. Hydraulic Piston Push
Vibratory	1. Offset Hammer - full supported mandrel* 2. Direct Hammer - offset mandrel* 3. Inside Mandrel with Enlarged Shoe
Jetting	1. Covered Mandrel with Outside Jets 2. Jet Probe with No Covering Mandrel
System Combination	1. Static with Vibratory* 2. Static with Jetting* 3. Vibratory with Jetting

* Currently used in the United States.

With few exceptions, all methods in Table 2-2 employ a steel covering mandrel that protects the PVD material as it is installed. All methods employ some form of anchoring system to hold the drain in place while the mandrel is withdrawn. Another common feature is that the PVD material comes in rolls and is threaded through the mandrel in a variety of ways. The major difference between the listed methods is in the technique used to insert the mandrel into the ground.

Commonly used methods employ an installation mast that contains the material reels, mandrel, and method of installation force. Added to this is a carrier, which is a crawler excavator or crawler crane, depending somewhat on the depth of installation. Specifications usually describe the acceptable method of installation(s) as static, vibratory, and/or static-vibratory.

Typical installation equipment and carrier pieces are illustrated in Figure 2-3.



Apevibro.com (top), U.S. Patent Office (bottom)

Figure 2-3. Typical prefabricated vertical drain installation equipment: installation rig (top), box mandrel (bottom).

The typical mandrel used in the United States has either a diamond shape of approximately 5.5 inches by 2.4 inches or a rectangular shape with dimensions approximately 5 inches by 2 inches. In some cases, especially with vibratory applications, a stabilizing fin will be necessary. In most installation masts, the PVD material is fed through the mandrel from a storage reel mounted near the base of the mast. The material travels up the inside of the mast, over a sheave at the top, and down through the steel mandrel.

The sequence of installation operations starts when the steel mandrel has been threaded with PVD material and attached to an anchor at the bottom of the mandrel. The anchor typically consists of a 0.5-inch diameter piece of cable or rebar or a special anchor plate made of sheet metal. The rebar and cable anchors will be approximately 8 inches long, and the sheet metal anchor plates will be made from a thickness sufficiently thin enough to allow the excess area to fold up around the steel mandrel, minimizing its cross-sectional area to slightly in excess of the cross-sectional area of the steel mandrel. A small "handle" is spot welded to the sheet metal anchor for insertion of the PVD material. When the sheet metal anchor is used, the PVD material is inserted through the handle and reinserted in the bottom of the mandrel.

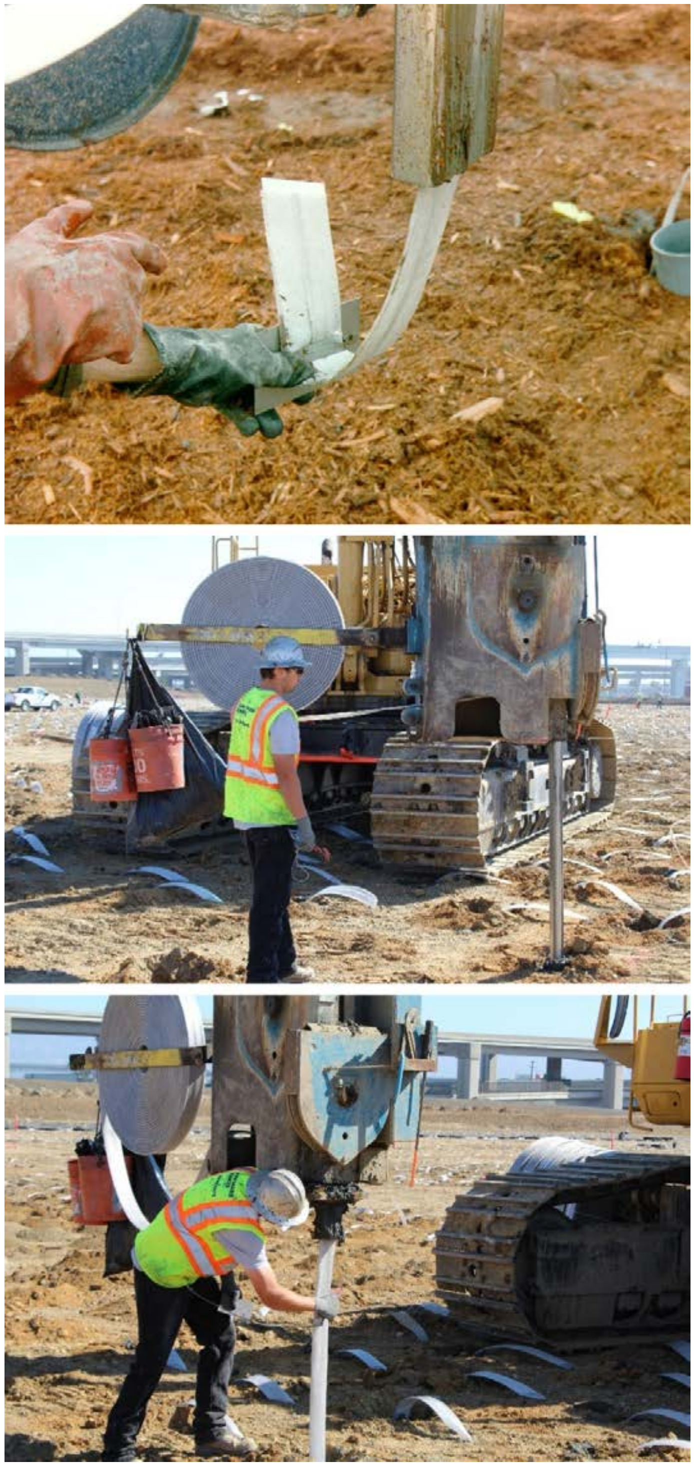
Once the anchor is in place, the mast is positioned over the location of the drain, and the mandrel, along with the PVD material, is inserted into the soil. During installation, the drain is completely protected in the steel mandrel from damage due to obstructions in the soil. When the PVD has reached its proper depth, the mandrel is withdrawn, leaving the PVD material in the ground. The mandrel progresses upward until the bottom of the mandrel is above the ground level. At this stage, excess PVD material is pulled through the mandrel to allow for sufficient "cut-off" or "stickup" above the working surface, and the PVD material is cut using hedge shears or other devices, and attached to another anchor. This procedure is then repeated at succeeding drain locations. A typical description of this procedure suggests a giant sewing machine. In fact, some of the installation masts are actually called drain stitches.

Except for unusual installation methods, where no mandrel is used at all, or where it is installed in sections, the length of the drain material must be slightly longer than the desired installation depth. Therefore, extremely deep drains (150 feet in depth) may require very large or specialized equipment. PVDs have also been installed from large barges on water and with amphibious marsh buggies.

Once the PVD material on the reel has been depleted, another is attached by splicing to the previous roll. The common method of splicing is to insert the core at end of the old roll into the new roll a minimum length of 6 inches. Both the cores and geotextile must overlap to maintain continuity of flow channel and filtration, respectively. At this point, at least 10 staples (4 on each side and 2 in the middle) will be used to hold the ends together. The splice is formed so that the bottom side of a vertical drain can be inserted into the upper end to ensure continuous flow.

Where obstructions are located above the compressible stratum and cannot be penetrated using normal procedures, pre-drilling or offsetting the drains will be necessary. Obstructions within the compressible stratum that cannot be penetrated using normal installation procedures can only be offset.

Normally, for PVD installation, a one-to-two-person labor crew is required to handle the drain preparation, cutoffs, changing of drain rolls, and anchor attachment. The only other labor required will be that to run the carrier piece and to perform occasional repairs to the equipment. The construction procedures are illustrated in Figures 2-4 and 2-5.



Courtesy Menard Group USA

Figure 2-4. Typical prefabricated vertical drain installation procedure: placing the anchor on the drain (top), inserting the mandrel into the ground (middle), cutting the drain after withdrawing the mandrel (bottom).

Courtesy Menard Group USA

Figure 2-5. Typical prefabricated vertical drain splicing procedure: inserting the drain core within the jacket to maintain continuity (top), stapling the drain splice (bottom).

3.1.1.1 Specialized Equipment

While not commonly used, there are situations that will require very specialized equipment to install the PVDs. Situations where this might occur include the following:

- Unstable working surfaces
- Sloped surfaces
- Subsoils that are very difficult to penetrate
- Extremely deep drains (150 feet or more in depth)

Some examples of unusual equipment were previously discussed. These include the need for special carrier pieces for extremely deep drains and the use of marsh buggies on unstable working surface situations. Other examples include special external jetting techniques mounted on lightweight skid platforms for use on steep slopes and the use of test boring equipment to install pipe drains with PVD materials for construction of relief wells in existing dams.

Layers that might require special drilling techniques are fills containing large amounts of rubble, concrete, old slabs or footings, buried riprap or large boulders, and any cemented layers. Normally, if the soil can be augered for pre-loosening, its cost should be included in the PVD installation. However, if it is anticipated that obstruction drilling, as described above, will be necessary, a special obstruction of pre-drilling pay item should be established.

3.1.2 Staged Construction

When embankments of moderate to large heights need to be constructed on soft foundation soils with low strength and high compressibility, excessive settlement and instability of the fill can occur if the fill is placed too quickly or too high. A solution to this problem is to build the fill in stages, or staged construction, such that the foundation soils consolidate and gain strength in between additions of fill material. The rate at which fill can be placed is related to the consolidation characteristics of the foundation soils. Monitoring of settlement and pore water pressures in the foundation soils allows prediction of when additional fill can be added without causing excessive settlements and/or slope instability. This process is generally used when time exists to allow the settlement to occur during the construction process. PVDs can be used with staged construction to speed up the consolidation process to reduce the waiting times between fill placements.

3.2 Materials of Construction

3.2.1 Prefabricated Vertical Drains

PVDs are relatively flat and approximately 4 inch wide by ⅛ to ⅜ inch thick. Types of PVDs are discussed in Section 1.2. The material generally consists of a plastic core formed to make channels and a loose geotextile cover. These two components are equally important in the function of a PVD.

The purpose of a core is to create low resistance flow channels in order for water to flow along the length of the PVD. In addition to providing the flow path, the core maintains the drain configuration and shape, provides support for the filter jacket, and provides the tensile and compressive strength of the drain. It is important that the core have a certain amount of strength and flexibility.

The function of the geotextile is to provide a surface that inhibits soil particles from penetrating into the core channels, while allowing passage of water into the drain. Its secondary purpose is to prevent closure of the interior core channels and to form the outside of the flow channels of the core.

It is important that the apparent opening size (AOS) of the geotextile be such that it allows only a few particles to penetrate but will not allow sufficient soil particles to clog the core. Experience has shown that a geotextile having an AOS in the range of 0.15-0.074 mm sieve size (U.S. #100 to #200) will be effective. Geotextiles within this AOS range have proven to be very successful for all projects.

The selection of the PVD type to be used is an important part of the overall design process, and the following information is intended to aid in this selection process. When selecting a PVD, the primary parameters include jacket filter permeability and characteristics, material strength, flexibility, durability, discharge capacity (q_w), drain resistance, and equivalent diameter (d_w). The filter should be designed to either prevent soil particles from passing through, or allow formation of a natural filter cake. The filter needs to resist tearing during installation, and the filter characteristics need to be maintained for the duration of the consolidation period. The discharge capacity of the drain under lateral stresses needs to be considered. Guidance is provided in Rixner et al. (1986a) and Chu et al. (2004). Buckling and crimping of the PVD is also a concern. It is recommended to assume a conservative maximum discharge capacity of 3,500 ft^3/year. Koerner (2012) also provides general guidance on geotextile filter and discharge characteristics.

Some procedures argue that the geotextile openings should be small enough to prevent soil particles from passing through and causing “siltation” and reducing discharge capacity. Other procedures believe that the openings should be large enough to allow for the formation of a natural filter. The well capacity used in design will depend on the volume of the drain opening, lateral earth pressure, folding and crimping of the drain, and the infiltration of fines into the PVD. Design criteria are presented in Holtz and Christopher (1987) for critical/severe conditions and for less critical/severe situations, the former having a more stringent design process. Projects with critical conditions include those where PVD failure will result in loss of life, significant structural damage, and/or the cost of repair exceeds initial installation cost. Projects with severe conditions include gap graded soils, high hydraulic gradients, dynamic flow, or reversing flow conditions.

Soil disturbance is most dependent on the mandrel size and shape, the soil macrofabric, and installation procedure. Soil disturbance can slow the rate of consolidation. Therefore, it is critical to consider the effects of a smear zone which results from soil disturbance, which can be minimized through proper mandrel selection.

3.2.2 Drainage Layer

The second component of a PVD installation (or any vertical drain project) is the drainage layer or method of conveyance and discharge of water from the drains. In most cases, this is accomplished using a drainage blanket consisting of a granular material. Synthetic drains, often called strip drains, can be laid horizontally along the installation surface and connected to each individual PVD to provide a clear drainage path for the pore water to the atmosphere, without creating any head loss. Often the strip drains are outletted in gravel ditch drains at the edge of the embankment.

Where sand or gravel is used, the drainage blanket can also serve as a working platform to help support equipment used for PVD installation. Ideally, sand should be clean and washed, such as concrete sand with the fraction minus the #200 sieve being less than 3 to 4 percent. The design of the drainage layer is based on its ability to transport the excess pore water without any head loss. If sand is used, it should have a minimum thickness of at least 1 foot, but more typically it is 1.5 foot to 3 feet. Where it is used directly over soft soils, typical designs require at least 1 foot of extra thickness because of potential contamination, or use of a geotextile separator (FHWA 2008). Where there is a possibility of mud waving, as much as a 3 foot thickness has been used. If gravel is used, it should also be very clean. Because of its permeability, gravel can be limited to a thickness of 8 inches, with geotextiles on both sides to keep it from being contaminated by intrusion from the lower soils or the upper embankment materials.

Where strip drain material is to be used in lieu of a drainage layer, there are several different techniques that can be utilized. Typically, the strip drain material is approximately 1 inch in thickness and either 0.5 or 1 foot in width. Depending on the flow rate of the individual strip drain and the length and number of drains attached to each strip drain, either the smaller or larger size may be necessary. Typically, a strip drain is attached to every individual drain, which means one horizontal strip drain for every row of PVDs. Occasionally; a single strip drain has been used for as many as three different rows, which requires that each PVD be connected to each strip drain. An example of a strip drain application is shown in Figure 2-6.

Courtesy Menard Group USA

Figure 2-6. Strip drains.

Consideration must be given to stability of the working surface. Often the thickness of the granular blanket must be increased to allow for support of the PVD installation equipment. An alternative is to reinforce the drainage blanket with geotextiles and/or geogrids. This may have a twofold effect: to provide a stable working surface, and to minimize the necessary thickness of the drainage layer due to contamination from the soils below. A combination of a working platform and drainage layer is often cost effective.

If PVDs are installed on uneven surfaces, such as on the sides of an existing embankment, the drainage layer effectiveness and stability must be considered. The working surface may have to be altered to allow for the installation of PVDs, such as a benching procedure that may disrupt the continuity of the drainage blanket. To ensure proper functioning, the drainage layer must be outletted.

4.0 DESIGN

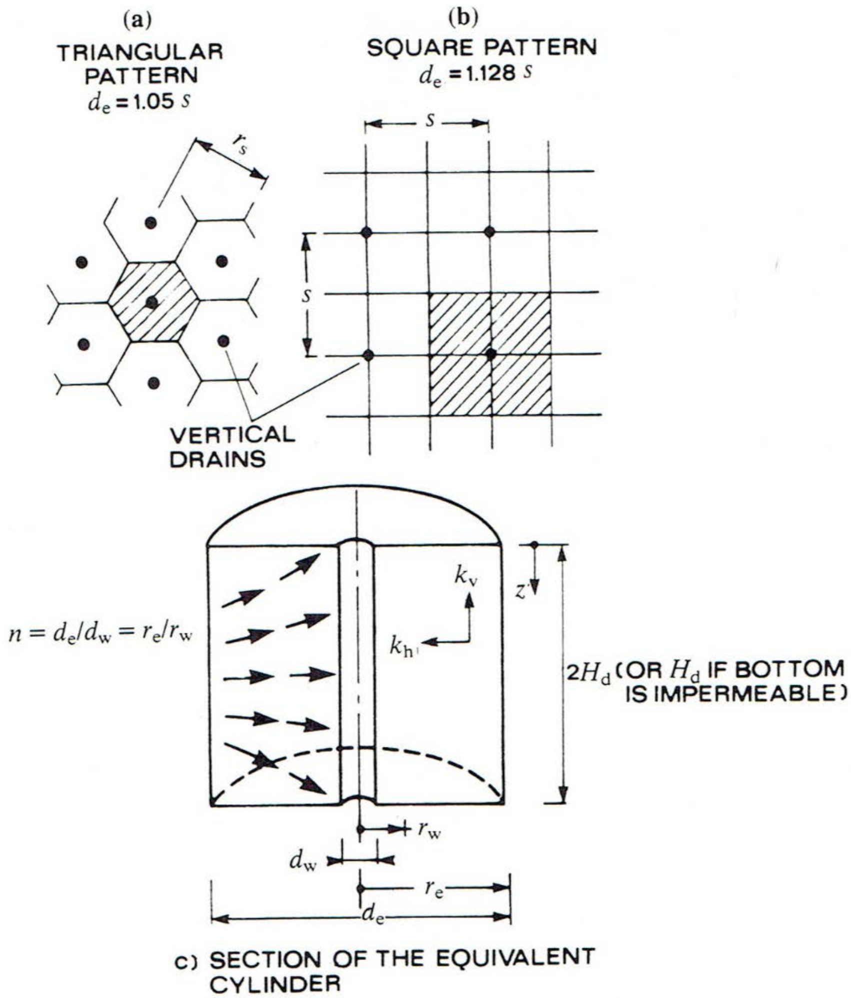
4.1 Design Considerations

The principal objective of soil preloading, with or without the use of vertical drains, is to achieve a desired degree of settlement in the site soils within a specified period of time. The design of vertical drains is primarily based on the use of fill or surcharge to preload the site soils and PVDs to reduce the length of the drainage path and, thereby, accelerate consolidation of the site soils. Vacuum consolidation (see *GeoTechTools*) may also be used as surcharge through an increase in the effective stress in the site soils. Increasingly, PVDs are being used for specialized applications such as liquefaction potential reduction where specialized knowledge is required. Such an application is beyond the scope of this technical summary and interested readers should see Rollins et al. (2004) and Rollins and Strand (2007).

Preloading site soils to bring about consolidation requires time for the dissipation of the excess pore pressures developed and for settlement of the soils to occur. The use of PVDs shortens the drainage path and speeds up the settlement over the case of surcharging without vertical drains. The proper design of a PVD installation requires knowledge of the type and extent of the foundation soils and their pertinent engineering properties. Engineering analyses must include, among other items, predictions of the amount and the rate of settlement, both during and after construction, and the embankment stability during all phases of construction. For consolidation analyses, the subsurface investigation program must define the extent and depth of the compressible strata and secure high quality undisturbed samples to determine past maximum pressures, coefficients of compressibility, and coefficients of consolidation, both in a vertical and horizontal direction. In addition, Standard Penetration Test (SPT) blow counts (N_{60}) is also significant in determining the installation costs and potential for the need for pre-drilling prior to placement of the PVDs. Piezocone penetration testing (PCPT or CPTu) is an excellent complement to SPT measurements, as it is fast, provides continuous profiling, and is particularly suitable for soft soils. For detailed discussions on SPT and PCPT the reader is referred to FHWA (2002).

The design of vertical drain systems begins with traditional settlement analyses to determine the total and the time rate of settlement under final project loads without the use of drainage measures. Settlement analysis of cohesive soils is conducted using Terzaghi's one-dimensional consolidation theory and many textbooks detail the procedures to follow (e.g., see Holtz et al. 2013) as well as design manuals (see NAVFAC Design Manual 7.1).

The design of all vertical drains systems is based upon consolidation theories. Vertical drains speed up the consolidation of a clay layer by introducing a radial drainage pathway into the soil strata that shortens the water pathway compared to vertical consolidation only. Although the use of vertical drains entails two-dimensional consolidation in the vertical and horizontal directions, generally only the horizontal consolidation is considered in practical cases, as the coefficient of consolidation in the horizontal direction (c_h) is usually greater than the coefficient of consolidation in the vertical direction (c_v) and the drainage path in the horizontal direction is much shorter than in the vertical direction. Thus the problem reduces to one-dimensional radial drainage. Barron (1948) developed the first radial drainage solution in English for vertical sand drains, and this remains the basis for vertical drain design today. Barron's work was based on the simplifying assumptions of Terzaghi's one-dimensional consolidation theory that the soil is homogeneous and completely saturated, the water and soil grains are incompressible, the flow of water and soil compression are in one direction only, Darcy's Law for water flow is valid, and the permeability and compressibility of the soil are constant over the increment of stress. Barron (1948) proposed the use of a triangular drain pattern (Figure 2-7[top left]) while Kjellman (1948) proposed a square pattern (Figure 2-7[top right]). In either case, the boundary conditions of the problem refer to an equivalent soil cylinder of diameter, d_c, having an impermeable outside surface and an inner cylindrical drain as shown in Figure 2-7(bottom) (Holtz et al. 1991).



Holtz et al. 1991

Figure 2-7. Drain patterns: triangular pattern (top left), square pattern (top right), equivalent cylinder (bottom).

Solutions to the radial consolidation problem are available in a number of publications including Barron (1948), Hansbo (1979, 1981, and 2004), Rixner et al. (1986a), Holtz et al. (1991), and Han (2015); and reference to these is made for more complete coverage of the theories. The solution of Rixner et al. (1986a) is used in the next section to detail the design procedure.

4.2 Design Procedure

The design of a PVD system consists of the selection of the type, spacing, and length of the drains to accomplish the required degree of consolidation within a specified time. The design process begins with the development of settlement analyses without PVDs to determine the total magnitude of settlement and the time rate of that settlement under the expected final embankment load. If the time to reach 90 to 95 percent of these projected settlements is too great and beyond the allowable contract construction time, the use of PVDs is a design solution to reduce the time for the settlement to occur.

The assumptions used in developing one dimensional consolidation theory have been applied to the development of radial drainage theory related to vertical drains, which resulted in the following relationship between time, drain diameter, spacing, coefficient of consolidation and the average degree of desired consolidation (Rixner et al. 1986a).

$$t = \frac{d_c^2}{8c_h}\left[F(n) + F_s + F_r\right]\ln\left[\frac{1}{1-\bar{U}_h}\right] \qquad \text{[Eq. 2-1]}$$

where:

t	=	time required to achieve desired average degree of consolidation
$\bar{U}_h$	=	average degree of consolidation due to horizontal drainage
d_c	=	diameter of the cylinder of influence of the drain (drain influence zone)
c_h	=	coefficient of consolidation for horizontal drainage
F(n)	=	drain spacing factor
F_s	=	Soil disturbance factor (smear zone)
F_r	=	Well resistance factor

The drain spacing factor F(n) is determined as follows:

$$\mathrm{F(n)} = \ln\left(\frac{d_c}{d_w}\right) - 0.75 \quad (\textit{simplified}) \qquad \text{[Eq. 2-2]}$$

where:

d_w	=	diameter of an equivalent circular drain

Note that Equation 2-1 does not consider consolidation from vertical drainage.

The following discussion of the components of Equation 2-1 should provide an understanding of the factors that can affect the design of PVD systems.

t

The time to achieve the required degree of consolidation ($\bar{U}_h$) for a pattern and equivalent spacing d_c, and drain diameter d_w.

$\bar{U}_h$

Normally, an average degree of consolidation of 90 to 95 percent is desired and, it is a function of the magnitude of post-construction settlement that the project can tolerate. It should be further understood that the contribution of vertical consolidation may be significant and should be considered for major/complex projects. Often it may approach 30 percent of the measured settlement.

d_c

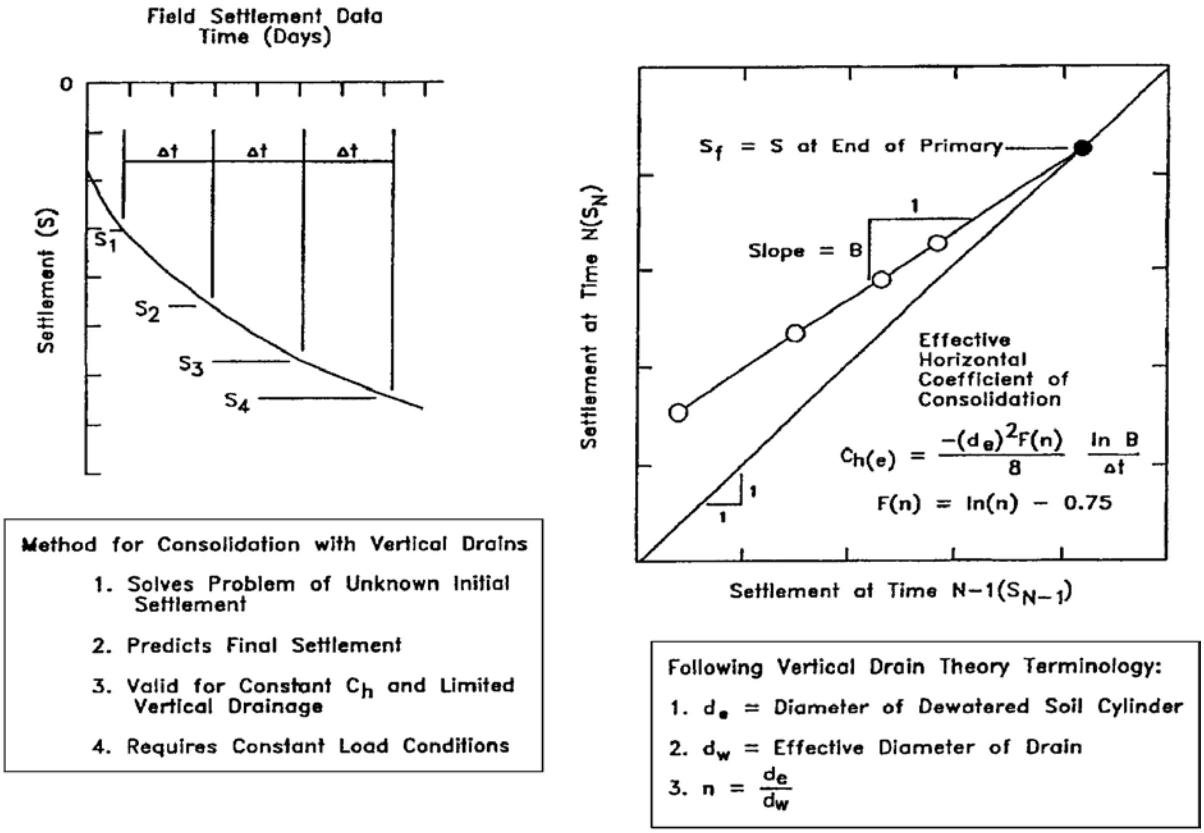
The diameter of the cylinder of influence of each PVD. When using an equilateral triangular pattern, d_c is 1.05 times the spacing between each drain. In a square pattern, d_c is 1.13 times the spacing between drains. Typically, to achieve approximately 90 percent consolidation in 3 to 4 months, designers often chose drain spacing between 3 feet to 5 feet in homogenous clays, 4 feet to 6 feet in silty clays, and 5 feet to 6.5 feet in coarser soils.

c_h

The horizontal coefficient of consolidation (c_h) of a particular layer can be obtained from laboratory consolidation tests. Even with proper laboratory techniques and high-quality undisturbed samples, the designer is fortunate to be within 50 percent of the actual coefficient of consolidation. Normally, only the coefficient of vertical consolidation (c_v) is obtained from standard consolidation tests from which the horizontal coefficient is estimated.

High quality laboratory tests have consistently shown c_h to be greater than c_v, the coefficient of consolidation in the vertical direction. A common conservative approach is to assume c_h is directly related to c_v, without direct measurement values. For design, c_h is generally taken as 1.2 to 1.5 c_v, if no or only slight evidence of layering is evident upon examination of the partially dried clay sample. If layering of silt and sand in discontinuous lenses is evident, ratios of 2 to 4 are indicated. For varved clays and other deposits containing embedded and more or less continuous permeable layers, ratios of up to 10 can be considered. The extent of layering in compressible deposits is best assessed by field PCPT testing. Additional guidance on the determination of c_h can be found in Rixner et al. (1986a). Relationships between the coefficient of consolidation in the vertical direction, c_v, and the liquid limit, can be used for preliminary design purposes (as shown in Figure 2-10 later in this section).

The most accurate assessment of the average c_h can be made from settlement data obtained from a test fill or from data obtained from the first stage in stage construction by using the Asaoka method (Asaoka 1978) shown in Figure 2-8 and discussed in Bartlett et al. (2001).



Bartlett et al. 2001

Figure 2-8. Asaoka (1978) method for determining c_h and end of primary consolidation.

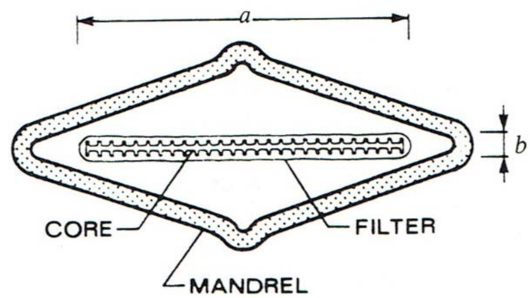
Equation 2-1 does not take into consideration any consolidation from vertical drainage. The vertical drainage effect may be minor when the compressible layer is over 15 feet in thickness, and for a conservative approach, no beneficial effects may be considered for vertical drainage in preliminary feasibility evaluations.

When several compressible layers of varying soil properties are encountered, care must be taken to evaluate each layer and its effect on the total consolidation. Settlement and time rate of settlement computations can be performed on each layer with fixed time and spacing to determine if sufficient consolidation will occur within a given time constraint. For a very conservative approach, the lowest c_h can be used to design the spacing. However, the magnitude of settlement should still be computed for each layer.

d_w

Hansbo (1979) suggested that when a band-shaped drain (PVD) is used, the equivalent diameter, d_w, should be a cylinder of having the same circumference as given by Equation 2-3 and shown in Figure 2-9. The PVD thickness is “a” and the PVD width is “b.”

$$d_w = \left[2(a+b)\right]/\pi \qquad \text{[Eq. 2-3]}$$



$d_w = [2(a+b)]/\pi$

Holtz et al. 1991

Figure 2-9. Typical cross-section of a band-shaped drain and mandrel.

There have been varying methods and recommendations to determine the equivalent circular drain size for a PVD. Various diameters ranging from 1.5 inch to 5.5 inch have been used, but the most common is to use 2.5 inch. In fact, there is little difference in design if 2.0 inch to 4.0 inch is used.

F_s, F_r

The above basic relationship can be modified further to consider the effects related to soil disturbance and well resistance. The effects of both soil disturbance and well resistance are often ignored for typical projects. Chu et al. (2004) provide guidance for when F_s and F_r should be considered in the design calculations. Additional information can also be found in Rixner et al. (1986a) and Holtz et al. (1991).

Considerations for Using Equation 2.1

Sample disturbance is typically ignored except for design in soils that are highly plastic, sensitive, and where the coefficient of consolidation has been accurately determined. Under these conditions, a sample disturbance factor $F_s \approx 2$ may be considered. Note that the effects of sample disturbance are more pronounced at drain spacing of less than 5 feet and by the contractor’s use of large, thick anchor plates.

Well resistance is rarely significant, and, therefore, ignored except for extremely deep drains or where a combination of high loads and very permeable soils is present. For guidance in these situations, the applicable reference is Hansbo (1979).

There are three basic variables that can be manipulated in order to achieve a desired result from Equation 2-1. These variables are time, PVD spacing, and surcharge. If the surcharge is increased, yielding greater settlement, then the PVD spacing can be increased in order to achieve the same amount of settlement in a given time (Fellenious and Altace 1999).

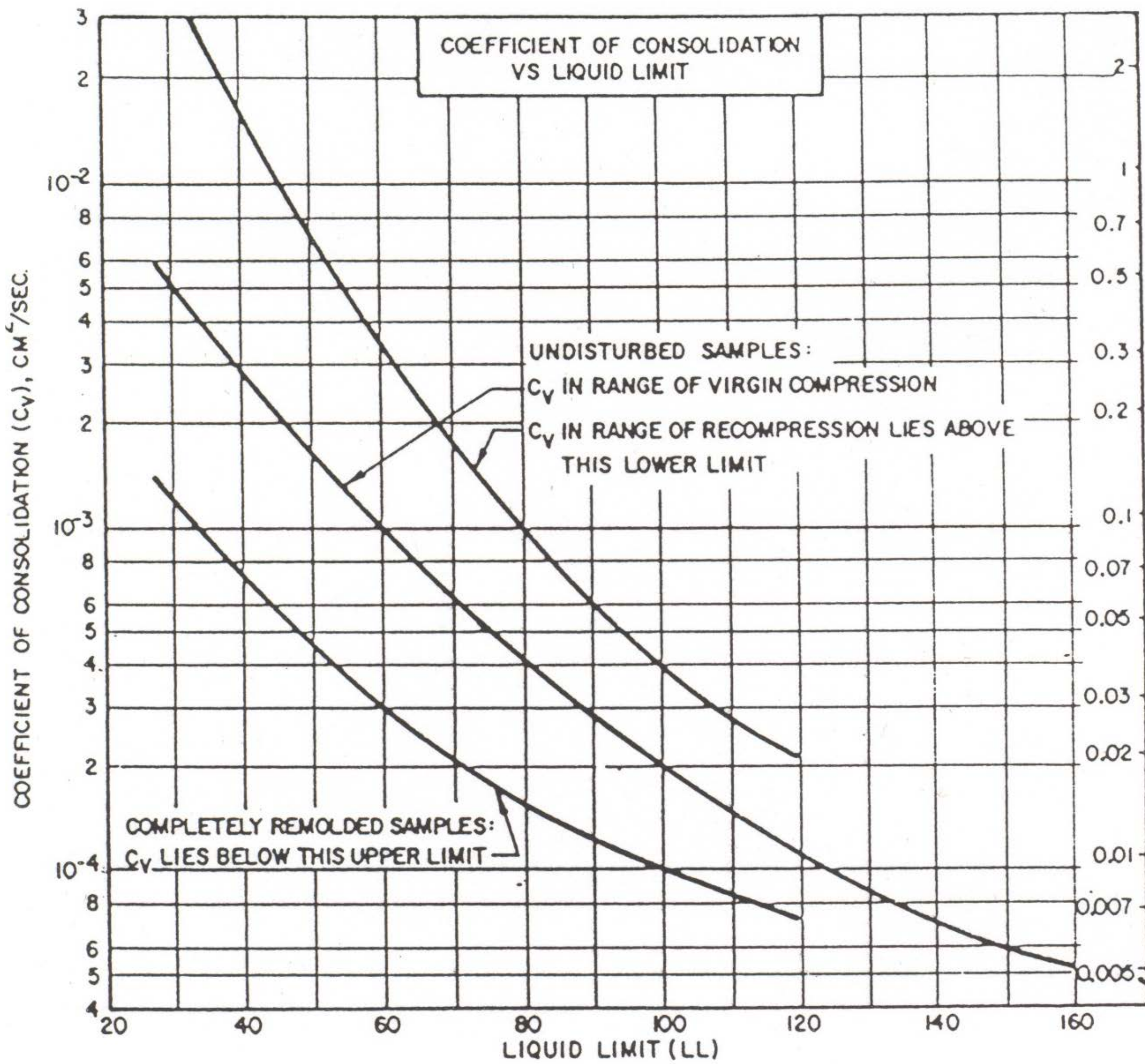
Another approach is to add surcharge in order to decrease the time required for settlement for the same spacing. Time can also be used as a variable, affecting the amount of surcharge or the PVD spacing. Using these variables, the designer can consider the cost of each, in order to determine the most economical solution.

Usually, time is a constant, with the designer varying spacing or surcharge in order to achieve the desired results.

For consolidation of organic soils, additional surcharge must be designed to obtain the required settlement attributed to secondary compression within the available time, i.e., overconsolidate the soil for the design embankment load.

4.3 Preliminary Spacing Design

Preliminary determination of PVD spacing using soil index data, such as liquid limits and project geometry, can be made. Project geometry reflects the area of loading, depth of soft compressible strata, and whether excess surcharge will be necessary. An approximate coefficient of consolidation, c_v, can be estimated from liquid limit values using the graph shown in Figure 2-10.



Department of the Navy 1982

Figure 2-10. Coefficient of consolidation versus liquid limit.

The center curve represents the case of virgin compression associated with good quality undisturbed samples while the lower curve is indicative of results from disturbed samples. The upper curve represents c_v for samples above the preconsolidation stress. Additional guidance on the selection of c_v from correlations and laboratory tests can be found in GEC 5 – *Evaluation of Soil and Rock Properties*. Direct determination of the horizontal coefficient of consolidation, c_h, is possible using piezocone dissipation test results. The procedure is detailed in GEC 5.

Assuming that the desired percentage of primary consolidation has been established or is in the 90 to 95 percent range, the designer can then either use computer programs available for vertical drain design, or solve Equation 2-1 to determine a preliminary spacing consistent with available time. A number of computer programs have been developed that perform such design procedures including FoSSA-Foundation Stress & Settlement Analysis (Adama Engineering 2003). Once the preliminary spacing is determined, the total project quantities can be computed from the project geometry.

4.4 Design Example

A highway embankment is planned at a location where available borings indicate a profile of 20 feet of normally consolidated clay with sand lenses over rock. Total settlement, based on one-dimensional drainage is estimated at 12 inches, with 90 percent occurring in 10 years. This estimate is based on preliminary evaluations of compression and consolidation coefficients obtained from classification data and liquid limit of 60.

The available construction time is on the order of 18 months; hence it is desirable to accelerate settlements by the use of PVDs to a point at which 90 percent of the total settlements would occur within about one year, without a surcharge load. To estimate feasibility and cost, a potential spacing must be determined.

From Figure 2-10, using the center curve, an estimated coefficient of consolidation (c_v) of 0.1 ft^2/day is obtained, and based on the existence of horizontal sand lenses in the profile, a coefficient of horizontal consolidation (c_h) of 2 times c_v can be assumed. Equation 2-1 can be solved for various drain spacings, using an equivalent diameter of drain of 2.5 inches to obtain the field time required to attain 90 percent consolidation. The solution using triangular spacing indicates required times on the order of 300 days at an approximate spacing of 8 feet or 500 days at an approximate spacing of 10 feet.

Note that this feasibility design method is only suitable for preliminary design or a quick check of a detailed design to determine if there are any errors. Since feasibility design is solely predicated on the liquid limit determined, it must be representative of the compressible stratum.

5.0 CONSTRUCTION SPECIFICATIONS AND CONSTRUCTION CONTROL

5.1 Introduction

PVD construction equipment, methods and materials, are well established with a proven track record of many successfully completed projects in the last several decades. The design concepts are well understood and the accuracy of the design is almost directly related to the accurate determination of the horizontal coefficient of consolidation. Therefore, method specifications are almost always used to implement this ground modification technology.

5.2 Specification Development

The role of good specifications is to reduce the problems, risks, and cost of PVD installations and to ensure an installation that achieves the intended objective of the design. To accomplish the above, agencies usually specify the general construction method and controls for PVD installation. Under this contracting method, the designer details the spacing, the extent and length of the drains, as well as the details of the drainage blanket (usually specified separately under earthwork items), and the horizontal discharge drains. End result specifications are not common since they would require extensive soil exploration and design by the contractor.

The specifications should provide the PVD installer as much flexibility as possible in achieving the intended result. The specifications should include quality control and assurance procedures. Initial installation trial drains should be required to establish standard procedures. At this stage of construction, such items as mandrel size, depth gauges, splicing procedures, verticality, and materials should be carefully inspected for compliance. Visual observations and periodic checks can then determine any variances or concerns once production has begun.

Inspection personnel should check the physical measurement of drain sizes (thickness and width) for compliance to specifications and variances from material submittals. If significant differences are noted, laboratory testing may be required to determine if PVD materials comply with specifications.

As part of the development of GeoTechTools, an extensive evaluation was made of specifications for PVDs. Eight specifications written by state DOTs, as well as the guide specification presented in the 2006 version of this manual, were reviewed and evaluated. These specifications were used to develop a guide specification entitled Standard Method Approach Specification for Prefabricated Vertical Drains and Fill Preloading that is intended to be a complete and fair specification containing commentary and instructions that is easily adaptable by the user for a specific project. This guide specification can be accessed on the

GeoTechTools website under the Prefabricated Vertical Drains and Fill Preloading Technology Information page. An additional specification resource is the recently published Wick Drain Guide Specification (Method), prepared by the Ground Improvement Committee of the Deep Foundations Institute (DFI 2014). This specification can be found by accessing the Ground Improvement Committee webpage on the DFI website (http://www.dfi.org/). Regardless of which PVD specification is used, it is recommended that the Hansbo (1986), Rixner et al. (1986a), Holtz and Christopher (1987), and Bo et al. (2005) references be reviewed to ensure that all PVD material and installation requirements are included.

5.3 Quality Assurance

One major advantage of the PVDs over other vertical drainage systems is the simplicity of field control. Once trial drains have been satisfactorily completed, inspection mainly consists of recording depths and locations of each drain, observing splices and verticality of equipment, taking occasional material samples for inspection and testing, and noting any major variances in procedure.

To ensure proper performance, the drains must be installed in accordance with the plans and specifications. It is important that field inspection personnel know the procedures and possible ramifications of any deviations.

The construction monitoring personnel should be thoroughly familiar with the contract drawings and specifications and should have a good understanding of the purpose of PVDs. The engineer in charge should have a good understanding of the total PVD solution, including site preparation, fill placement, and other items that might influence the performance of PVDs. Inspection of the PVD installation after initial procedures are established can become quite repetitive and monotonous. However, when changes or variations occur, these should be discussed immediately with personnel familiar with the project design requirements.

Other items of significance in construction, such as the drainage blanket material quality, embankment placement and compaction, and surcharge loading rate, must also be monitored.

5.3.1 Site Preparation

Site preparation includes any excavation and grading to prepare the site for installation of PVDs. This may include site clearing, excavation and/or filling operations to bring the site to grade, and construction of a working platform or drainage blanket. It is also important that the grade be such that a drainage blanket remains continuous and surface drainage will not erode the blanket to the extent where it is no longer continuous.

During construction of a working platform or drainage blanket, the field inspection personnel should be monitoring for any unusual soil movements, which would be indications of mud waving or a potential failure. On some projects, working mats or drainage blankets were installed too thick, creating failures, which were very expensive to repair. In one example project, a contractor installed a 5 feet haul road over a 2 feet drainage blanket prior to PVD installation in order to have access to a piling location. This haul road caused an extensive failure of the entire area that was to be stabilized.

5.3.2 Prefabricated Vertical Drain Material and Installation Equipment

Once the equipment and materials are checked for compliance with the specifications, the remaining field observations are rather simple. The major items to be monitored are verticality, depth of installation, and location. Most projects will provide an estimated depth or elevation across the site. It is routine to anticipate local variations from this estimate. However, the depth of PVDs should vary only slightly from that of adjacent drains. The inspector should refer to pertinent borings for a guideline to determine if proper depths are achieved or if there is a significant variance in soil conditions.

5.3.2.1 Material

Prior to installation, the PVD material should be visually observed to ensure that it is similar to that originally submitted and tested. The core and filter jacket should be continuous and comply with required dimensions, and the materials should not have been damaged during handling or storage.

Many specifications list intricate tests to be performed on the PVD materials. Many of these tests are taken from manufacturer's data and quite often are far beyond what is necessary. However, an individual project may have specific criteria, which are dependent on some stringent design or performance requirement.

5.3.2.2 Equipment

Inspection personnel should determine that the equipment complies with specification requirements. Some of the important items to be checked are the following:

- Penetration method
- Mandrel shape, size, and stiffness
- Anchor type and size
- Method to measure and determine penetration depth

- Method to measure and record installation force
- Means and procedures for pre-drilling, where necessary

Installation of trial drains to evaluate the installation equipment is recommended for all projects. A representative of the owner and inspection personnel should be present during the trial drain installation.

Variations in installation procedures, which might be necessary to penetrate to the required depth, should be evaluated during the trial program. Obstructions encountered are usually offset where possible or predrilled or removed if they encompass a significant area.

5.3.2.3 Submittals

Most PVD specifications require several submittals for approval. Submittals include the type of PVD material, the material specification sheet, and the source. In addition, a method and sequence outlining the installation procedure is often required. The material submittal quite often is either related to specific specification requirements or manufacturers' tests.

The installation submittals should address the following:

- Size, type, weight, maximum pushing force, vibratory hammer rated energy, and configurations of the installation rig
- Dimensions and length of mandrel
- Details of PVD anchorage
- Detailed description of proposed installation procedures
- Proposed method for splicing drains

A common additional requirement is the installer's experience. Many specifications require a minimum of three successful projects. This can be typically fulfilled either by the installation contractor's project experience or the project personnel's past experience.

Specifications should not require the delivery of PVD materials prior to the arrival of the installation equipment, as this requirement would entail two mobilizations by a specialty contractor. Instead, material certification should be required from the manufacturer in advance of mobilization.

5.4 Instrumentation Monitoring and Construction Control

Field instrumentation, such as piezometers, settlement platform and gauges, and inclinometers, are used to monitor performance of the PVDs and possibly control the rate of construction of embankment and/or surcharge. It is important that both the designer and the instrumentation personnel have a full understanding of the particular piece of instrumentation being installed and the strata in which the instruments are being placed.

Settlement measuring devices, whether platforms, deep settlement points, or horizontal deflection devices, are used to measure only the rate and total amount of consolidation. Inclinometers are used to measure horizontal deflection with depth and to warn against potential stability failure. The pore pressure devices (piezometers) are used for both calculation of achieved consolidation rate and to quantify excessive build-up of pore pressure that is an indication of potential failure. One caution concerning pore pressure devices is that there have been a significant number of projects where the rate of settlement has not agreed with the rate of pore pressure dissipation. In such situations, settlement data should be given priority as indicators of the rate of consolidation.

The proper selection of instrumentation devices and the frequency of monitoring during a project are important. For simple projects where stability is of no concern, and time is not the critical factor, only surface settlement platforms, which are relatively easy to install, are used. In situations where stability is critical, pore pressure measurements and measurements of horizontal deformations are also necessary. Where stability is of concern daily readings may be necessary both during loading and for the first few weeks after loading.

The inclinometers should be installed at the toe of the embankment or in front of retaining walls, with settlement plates and piezometers beyond the crest of the embankment and/or near the centerline. For projects where stability is of concern, the key tool in evaluating stability is the measurement of the displacement ratio, D_R, defined as the maximum cumulative horizontal displacement divided by the cumulative settlement determined from an adjacent settlement plate. Significant increases of displacement ratio with time or load can indicate when the foundation behavior is changing from a relatively stable condition associated with consolidation settlement to a state of excessive plastic deformation. Ladd suggested that when fill is placed relatively rapidly, a D_R approaching 0.4 during filling may be measured. This appears to be the upper limit for a stable foundation in which vertical drains have been installed (Ladd 1991). Displacement ratios of 0.2 generally correlate with a slope stability FS of 1.3 or greater.

For stable foundations D_R should decrease after the completion of filling. A typical plot of D_R vs time as developed for a multi-stage fill near the Maryland abutment of the Woodrow Wilson Bridge is shown in Figure 2-11.

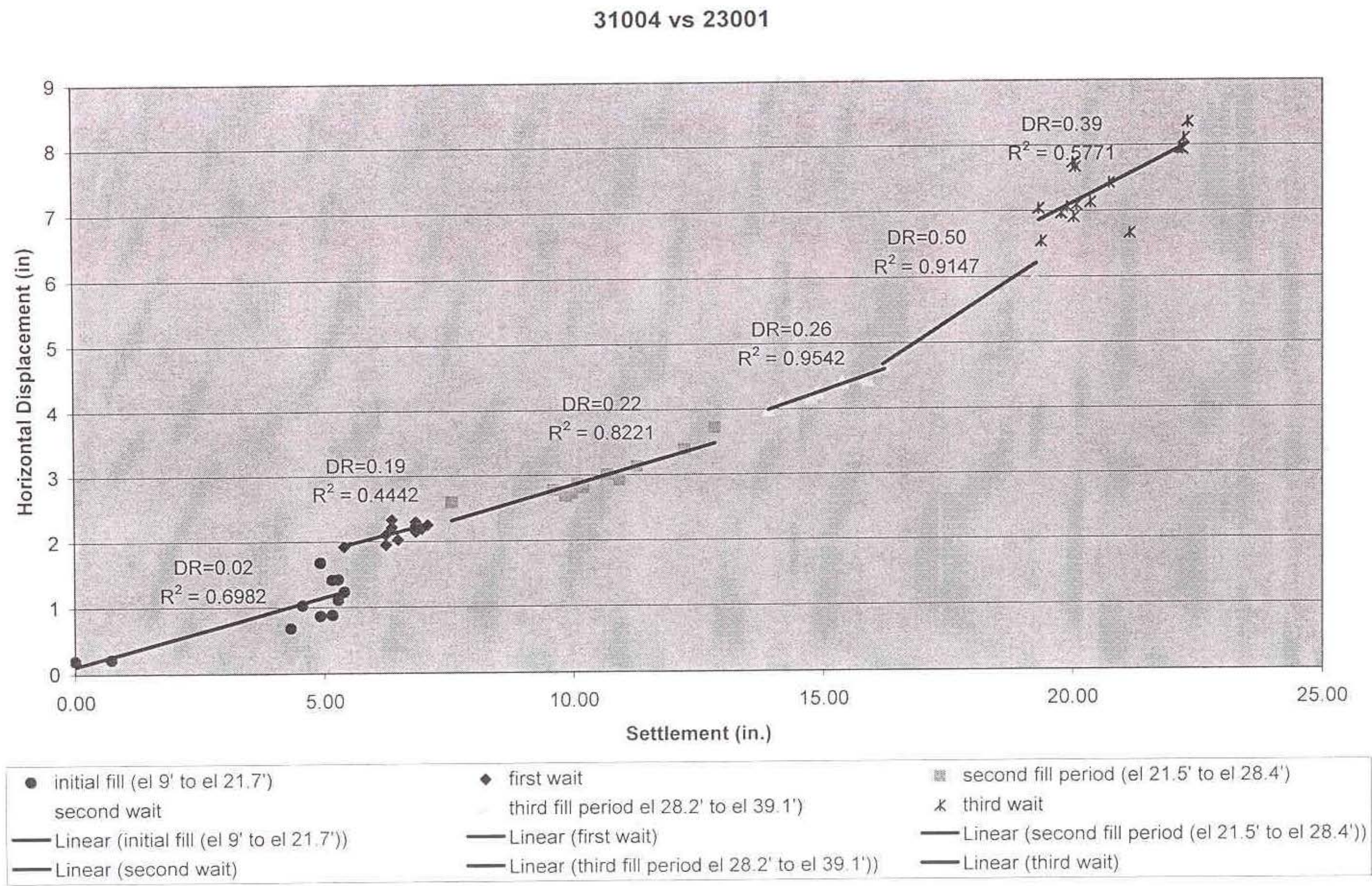


Figure 2-11. Displacement ratio versus time and stage filling.

In addition to monitoring displacement ratios, the inclinometer data should be plotted as incremental displacements as a function of time and depth. The incremental displacements will clearly show the depth at which horizontal strain is occurring and, therefore, the location of potential failure plane(s). The maximum horizontal displacement can be related to soil strain as well, as a further check on stability. Cumulative and incremental horizontal displacements, at the location where the displacement ratios in Figure 2-11 have been computed, are shown in Figures 2-12 and 2-13.

End of primary consolidation (EOP) can be obtained by the Asaoka method of analysis from settlement data. This method consists of plotting settlement on a time versus settlement plot of equal time steps as shown in Figure 2-8. Values of cumulative settlement at the start of a time step are plotted with respect to the settlement at the beginning of the next time step. When the settlement has not changed during a time step, primary consolidation is essentially complete. Details of the analysis and limitations are discussed by Bartlett et al. (2001).

It is usually best to install settlement platforms and other instrumentation after the installation of the horizontal drainage layer and PVDs. This allows for an unimpeded site during PVD installation and the ability to locate the most critical areas for instrumentation, especially for piezometers. If piezometers are installed prior to PVD installation, it often becomes difficult to locate the drain at equal distance from the piezometers' locations, especially where multiple piezometers are installed at different elevations. On some projects it might be valuable to install some instrumentation prior to PVD installation. The resultant information may be useful for interpretation in critical situations. When stability is a major concern, slope indicators and some piezometers and settlement devices should be installed prior to PVD installation. A typical layout with a complete set of instrumentation is shown in Figure 2-14.

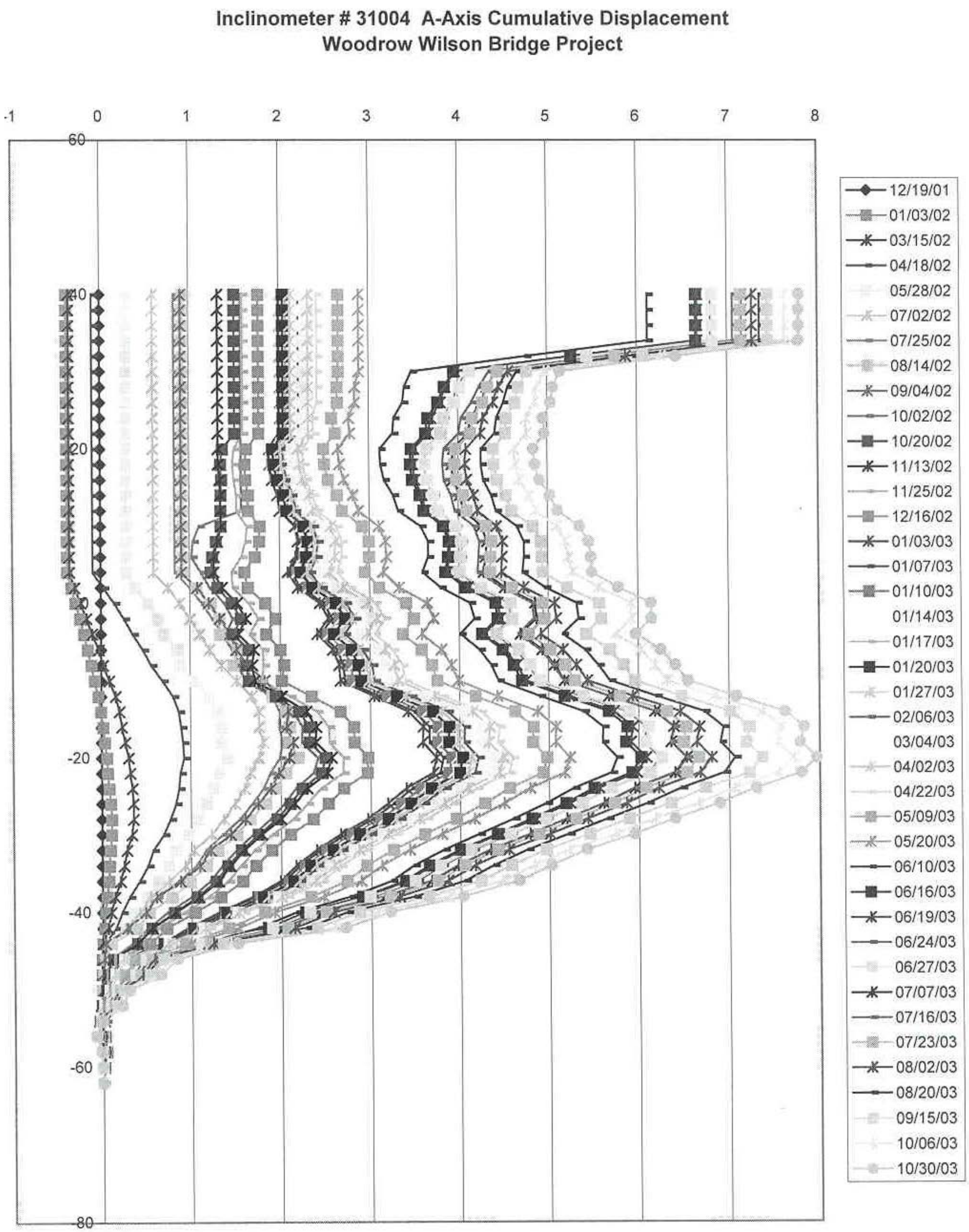


Figure 2-12. Cumulative displacement versus depth.

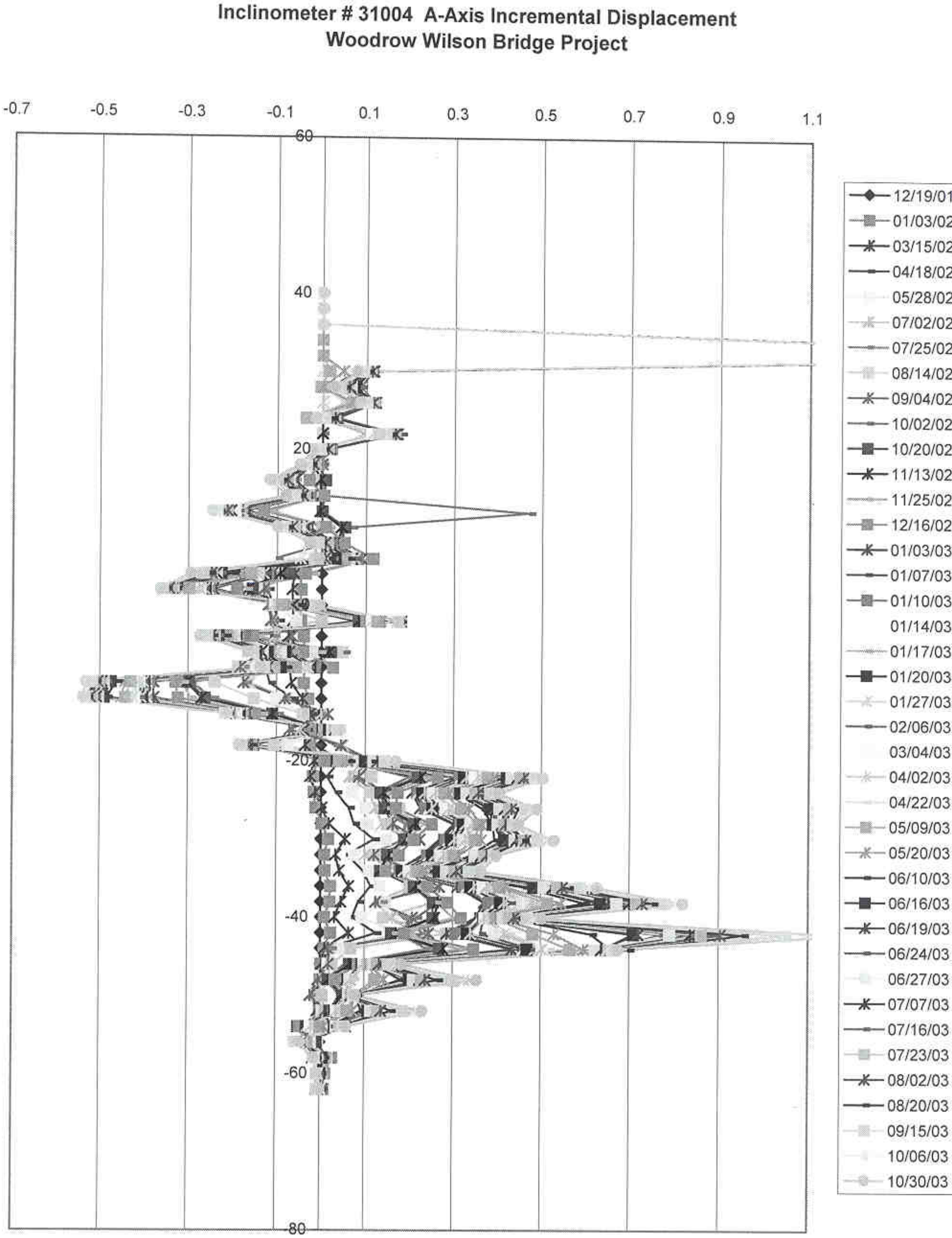


Figure 2-13. Incremental displacement versus depth.

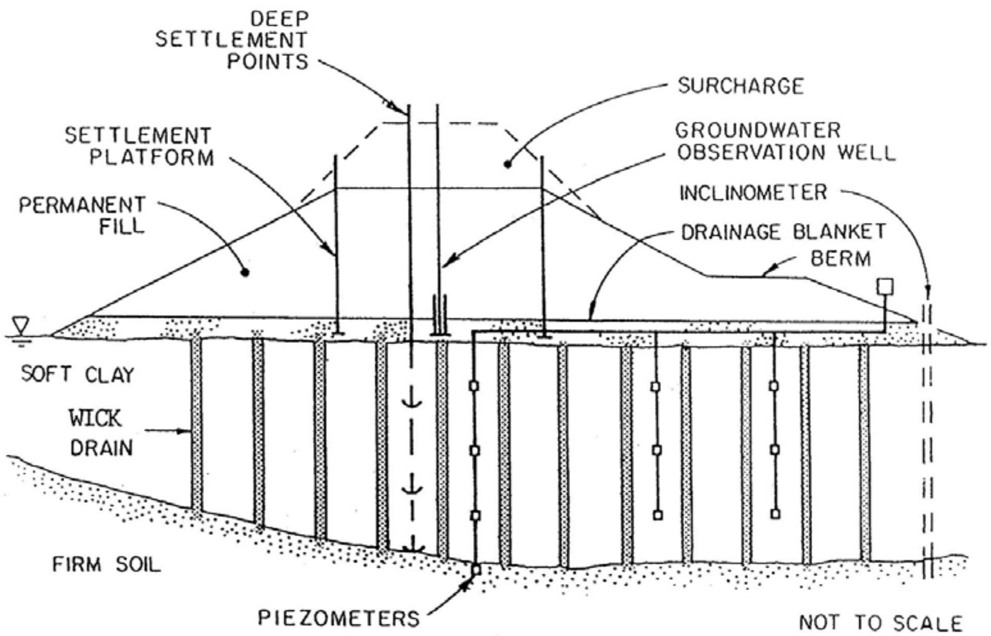


Figure 2-14. PVD Installation showing typical instrumentation and monitoring.

6.0 COST DATA

6.1 Cost Components

Often when estimating the cost of a PVD project, only the unit cost of the installed PVDs is considered, rather than the total cost including other ancillary items. Other factors affecting the total cost of the PVD solution include the following:

- Project Size, Topography
- Obstructions, Dense Soils
- Installation Methods
- Allowable Construction and Consolidation Time
- Allowable Post-construction Settlement
- Preload and Surcharge
    - Type and Material Available
    - Reuse of Material
    - Amount of Surcharge
- Drainage Blanket or Horizontal Drainage Path
- Design, Instrumentation and Monitoring
- Unit Cost of Installation

Note that the unit cost of installation, whether including the mobilization or not, is only one of many factors that affects the total cost of a PVD solution.

Some of the above factors are very difficult to quantify, and their costs may be included in other bid items. The following discussion will focus on all cost items since they have an important impact on the total cost.

- *Project Topography:* The project topography can play a role in the cost of a PVD solution. For example, it is very difficult, if not impossible, to install PVDs on significant slopes. If there is a great variation in grade on the site, significant earthwork may be necessary to provide a somewhat level working surface for installation.
- *Obstructions, Dense Soils.* The cost of PVDs increases significantly where soils are difficult to penetrate. Therefore, it is important to correctly identify the soils to be penetrated and the need to achieve specified depths. Depending on methods of

installation, the designer should be concerned with very stiff layers (generally N values of 15 or greater), or possible obstructions, such as boulders, rocks, previous foundations, known underground utilities, etc.

- *Installation Methods.* The installer should be given latitude in choosing the proper type of equipment for installation. This will ensure the most economical solution when difficulties arise. A specification that allows static, static vibro, and vibro-type installation usually results in the most economical solution. Jetting should be allowed only with the approval of the engineer and where it can be shown not to have an environmental impact. Impact methods should not be allowed, except for pre-drilling through non-compressible soils.

- *Consolidation Time.* Time required includes the estimated consolidation time, plus time for installation of the PVDs and for construction of the embankment and/or surcharge. Additional factors that affect time are the placement of instrumentation, site preparation, and drainage blanket installation. In general, the total time is the least variable factor because of project constraints.

- *Preload and Surcharge.* Preload is defined as the amount of fill material necessary to bring the site to the final elevation. This includes the amount of additional embankment necessary to accommodate the final amount of settlement. Surcharge is the added fill above final elevations that is used to accelerate settlement or minimize secondary consolidation. In many cases, the maximum amount of surcharge is controlled by the potential for slope instability.

  Time, surcharge magnitude, and PVD spacing are the significant variables in the total cost, and these variables can be optimized in the design stage to determine the lowest total cost. If time is the critical factor, either the PVD spacing must be closer together or additional surcharge must be placed in order to achieve the desired settlement within the specified time.

- *Drainage Blanket or Horizontal Drainage Path.* The availability and cost of sand, or other granular material, can often determine whether a granular blanket will be more economical than a geosynthetic strip drain solution. Quite often the granular blanket may be necessary for a working platform, even though it would be more expensive than the strip drain solution. The actual cost of a granular blanket is the difference between the granular blanket material and the local embankment material, whereas the strip drain material will be a totally added cost.

- *Design, Instrumentation, and Monitoring.* These items depend on the complexity of each project, but they may be a significant cost, especially if staged construction is required.

6.2 Summary of Typical Projects

Typical cost of PVD installations can be divided into three categories: small – up to 50,000 lineal feet, medium – 50,000 to 300,000 lineal feet, and large – greater than 300,000 lineal feet.

Typical unit price ranges are listed in Table 2-3 for projects where the soils do not present major difficulty in penetration, do not require special equipment or are not at unusually difficult sites.

Table 2-3. Typical Unit Price Ranges for PVDs

Site Category	Unit Price Range
Small	\$0.70 to \$4.00 per lineal foot
Medium	\$0.50 to \$1.00 per lineal foot
Large	\$0.30 to \$0.50 per lineal foot

Usually added to these costs is a mobilization charge of \$15,000 to \$25,000 per rig, as well as the cost of the drainage blanket and instrumentation. The unit cost could be significantly higher in areas of severe weather, labor shortages, or difficult site conditions, and where there is difficulties with installation of the PVDs.

7.0 CASE HISTORIES

The following three case histories illustrate successful projects.

7.1 Council Bluffs (IA) Interstate System Improvement Program

7.1.1 Project Description

The Council Bluffs Interstate System Improvement Program was a comprehensive interstate redesign that modernized the highway system and improved mobility and safety of approximately 18 miles of interstate. The I-29/US 275/IA 92 (I-29/US 275) Interchange reconstruction was completed between 2014 and 2016. The project consisted of removal of the existing bridges and portions of the existing roadway embankments and the construction of new roadway embankments, walking trail, bridge structures and other features. Six bridges were reconstructed and roadway improvements included raising the highway grades and expanding the lanes. Construction also included the consolidation of railroad operations into a new common corridor.

7.1.2 Subsurface Conditions

The site is located in the Missouri River valley floodplain. The subsoils consisted of silty clay and sandy silt overlying poorly graded fine to coarse grained sand of medium density. The upper 15 feet of the silty clay consisted of interbedded lean clay (CL) and fat clay (CH) with SPT blow counts (N_{60}-values) ranging from 4-10. The lower portion of the silty clay was approximately 20 feet thick and consisted of fat clay (CH) with SPT blow counts ranging from 4 to 6. The silty clays had moisture contents of around 40 percent and void ratios around 1.2. Below the silty clay, silt with sand and poorly graded sand was present, generally at depths of about 35 feet.

7.1.3 Design Concerns

The alluvial nature of the site soils led to design concerns about settlement and stability of roadway embankments. Post-construction settlement stemming from the added fill, to maintain the proper grade for the roadway embankments, led to settlement projections varying up to 30 inches, depending upon the height of fill placed. Various solutions, such as removal and replacement, surcharging, surcharging with PVDs, and use of stiff columns to transfer load were investigated. The high cost of removal and replacement eliminated this option. The schedule of the project and the time rate of settlement of the subsurface soils eliminated the option of just building the embankment and waiting for the settlement to occur. This led to the use of PVDs to accelerate the settlement, combined with surcharge in some areas, and the use of stiff columns in other areas.

7.1.4 Design

In areas of PVDs and surcharge, the results from laboratory testing indicated that a maximum total settlement of 24 inches would occur under approximately 40 feet of fill. Based on consolidation coefficients, it was determined that a 4-foot triangular pattern spacing of the PVDs would result in a settlement time period of approximately 3 to 5 months, once full height was attained. The PVD design depths ranged up to 35 feet. There were three areas to be treated using PVDs and fill loading. The estimated amount of PVDs was 18,400 in number with a linear estimated length of 690,750 feet, as shown in Table 2-4.

Table 2-4. Estimated and Installed PVDs

Area	Estimated Linear feet	Estimated No. of PVDs	Installed Linear feet	Installed No. of PVDs	No. of Days to Install
3	231,000	6,600	129,494	5,850	9
2	374,000	9,350	234,192	8,111	13
8	85,750	2,450	74,391	2,367	4
Total	690,750	18,400	438,077	16,328	26

Source: Menard Group USA

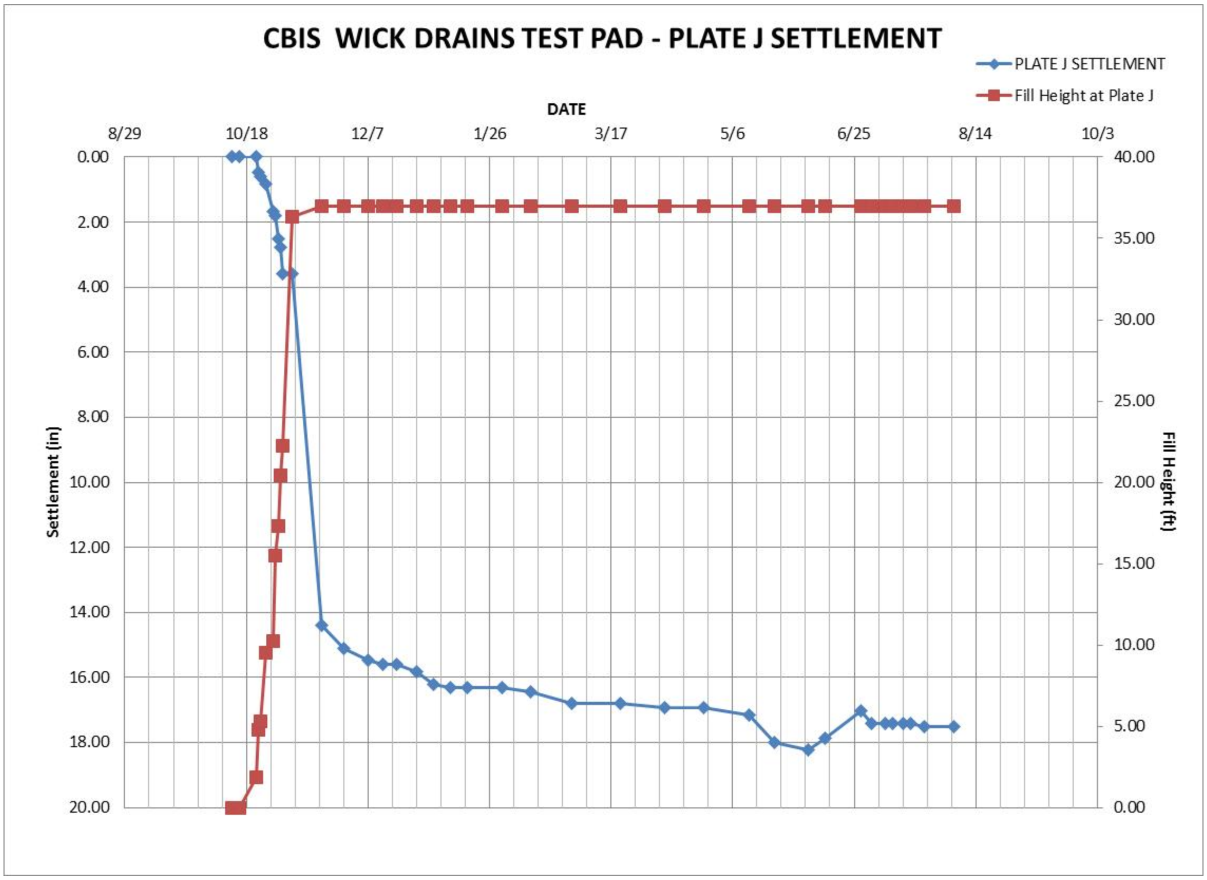
A test pad was constructed as proof of concept. Typical test pad results are shown in Figure 2-15, which indicate approximately 18 inches of settlement under 36 feet of fill. The majority of the settlement occurred in the first two months.

7.1.5 Project Results

The wick drain material (PVD) used was COLBONDDRAIN CX 1000®, 4.0 inches in width with 1/8 inch thickness. Installation was accomplished with either an ALIMAK® chain driven stitcher or an APE sprocket driven stitcher using a 5-1/2 inches by 3 inches rhombic-shaped mandrel. Installation of the PVDs in the three areas took 26 days to complete. As shown in Table 2-4, the installed numbers of PVDs and the total lengths were slightly less than the estimated values. The average installed length was about 27 feet. Actual settlements ranged from 4 to 20 inches and were generally attained in 2 to 3 months; well within the design time of 6 months.

7.1.6 Project Cost

There were four bids for this project, ranging from $476,465 to $823,396, to include the PVD installation and construction of the fill embankments. Subcontractor prices for the PVDs were approximately $0.50 per lineal foot. Mobilization costs for multiple rigs were bid at $40,000.



Courtesy of CH2M-Hill

Figure 2-15. Settlement versus fill height for test pad.

Acknowledgments **–** Iowa Department of Transportation, CH2M-Hill, Menard Group USA

7.2 Approach Ramps for Bridge Replacement

7.2.1 Project Description

This project involved the replacement of the existing Tifft Street Bridge over railroads located between the intersection of the Tifft Street with NY Route 5 ramps and Hopkins Street. Replacement of the existing bridge required construction of a new structure and embankments, incorporating the newest design standards, including greater vertical clearances over railroads and increases in lane widths.

The new structure would be significantly shorter on the west end, meaning the new west approach embankment would extend 300 feet beyond the old abutment. The total alignment of the new embankment centerline and bridge would be approximately 70 feet north of the existing centerline. In addition, the existing bridge had to remain open until the new bridge could accommodate at least two lanes of traffic.

7.2.2 Subsurface Conditions

The generalized subsurface soil profile indicated a 7 to 16 foot fill layer of cinders and slag overlying 2.5 feet of peat (under the west embankment only), 33 to 46 feet of soft silty clay, and a 10 foot layer of sand on top of limestone bedrock. The groundwater table was within 1.5 to 3 feet of the existing surface.

7.2.3 Design Concerns

Construction of the new approach embankments, varying in height from 15 to 35 feet, would result in considerable settlements. The west approach embankment would result in settlements ranging from 24 to 36 inches without treatment. The settlement was expected to occur over a 6-year period after the full fill height was achieved.

On the east embankment, the proposed fill would incorporate much of the existing fills. Therefore, settlements of only 12 to 24 inches were expected, but the time of the settlement would be similar to the west approach. The designers used PVDs, on both approaches, to accelerate the settlements to within a 6-month time frame.

On the west approach, a significant portion of the embankment south side slope would be under the existing bridge. It was felt that PVDs would be necessary in this area to prevent significant differential settlement of the side slope in future years. Therefore, some of the PVDs would have to be installed to maximum depths of 55 feet with a headroom clearance of only 26 feet.

On the east embankment there was no need for PVDs under the existing bridge, since the embankment did not extend past the old abutment. However, there was still need for PVDs adjacent to the existing embankment. Borings indicated that some PVDs in the new east embankment would have to penetrate slag under the existing embankment. The depth of the slag was 15 feet, and it would be difficult to penetrate, as indicated by very high "N" values in excess of 100 in some locations. The slag material was very abrasive and would be difficult to drill.

Analyses indicated that side slopes of 2H:1V could be safely constructed to a height of only 28 feet without foundation treatment. However, after the strength gain as a result of PVD accelerated consolidation, embankments could be constructed to the required heights of 35 feet.

7.2.4 Design

The west embankment was expected to settle up to a maximum 36 inches under maximum fill height, and the east embankment 24 inch maximum. A surcharge height of 36 inches above final grade elevation accelerated settlement. In the area of maximum settlement and where stability was of concern, the PVD spacing was 4 feet in a triangular pattern. In areas of lesser settlement, and where stability was not a concern, the pattern spacing was 7 feet.

The total quantity of the original estimate was 229,600 lineal feet of PVDs. Added to this quantity was the cost of 6,500 yd^3 of a granular material for the drainage blanket and 19,100 yd^2 of geotextile. New York State DOT estimated the total cost of the PVD solution, including auxiliary items, to be in excess of $800,000.

7.2.5 Project Construction Procedure

PVD construction began on the east side, and it became readily apparent that special drilling would be necessary to penetrate the slag fill on the area near the existing embankment. Special air rotary drills were used to pre-drill, and the PVD installation unit had to follow close behind. In areas farther from the existing embankment, a combination unit of vibro and static force was used to install the PVDs without pre-drilling. Approximately 82,000 feet of PVDs were completed on the east side within 4 weeks.

PVDs underneath the bridge on the west side were started simultaneously and completed in segments. A special sectional mandrel was developed such that it could be pinned together in a fairly rapid manner. The initial section was 20 feet long, with subsequent sections of 16 feet. At these locations, the holes had to be predrilled through a miscellaneous fill surface either with an auger or a special air track machine. After several days of trial procedures, a maximum production rate of 25 drains per day was achieved. This compared to a maximum production rate of more than 300 drains per day on the locations not under the bridge.

The PVDs on the west side that were not under the bridge were completed in 4 weeks. It was also necessary to angle some drains under high tension wires in order to achieve full coverage of the area. The total quantity of PVDs installed on the west approach was approximately 110,000 feet.

To install the drains in sections underneath the bridge there had to be more than one splice per drain. This requirement conflicted with normal specifications, which require only one splice per drain.

7.2.6 Project Results

Typical piezometers and settlement data from both the east side and west approach are shown in Figures 2-16 and 2-17.

Total maximum settlement of the west approach was 28 inches; on the east approach, the maximum settlement was 24 inches. The settlement was achieved in both locations in approximately 4 to 5 months and agreed fairly well with predictions. Piezometers, slope indicators, and settlement plates were used to determine the time rate and amount of consolidation. The purpose for the slope indicators was two-fold. While stability was not believed to be a concern, the factor of safety was marginal. Secondarily, it was believed that lateral movements could be used to determine the immediate settlement component.

7.2.7 Project Costs

This project was somewhat unique in that the use of several different installation units had to be bid under one item. The drains were installed using standard static machines along with pre-drilling in some locations, static vibro machines in other locations, and a special installation machine underneath the bridge.

The general contractor's bid prices were for (1) PVDs at $2.10 per foot; (2) granular drainage blanket at $17.00 per cubic yard.; and (3) geotextile at $1.40 per square yard. Using the original cost estimate, the actual prices resulted in a PVD solution less than $600,000, which compared quite favorably to projected costs.

Subsequent to this project, New York State DOT added a new bid item to account for PVD mobilization costs. This bid item attempts to account for the need for special equipment on projects.

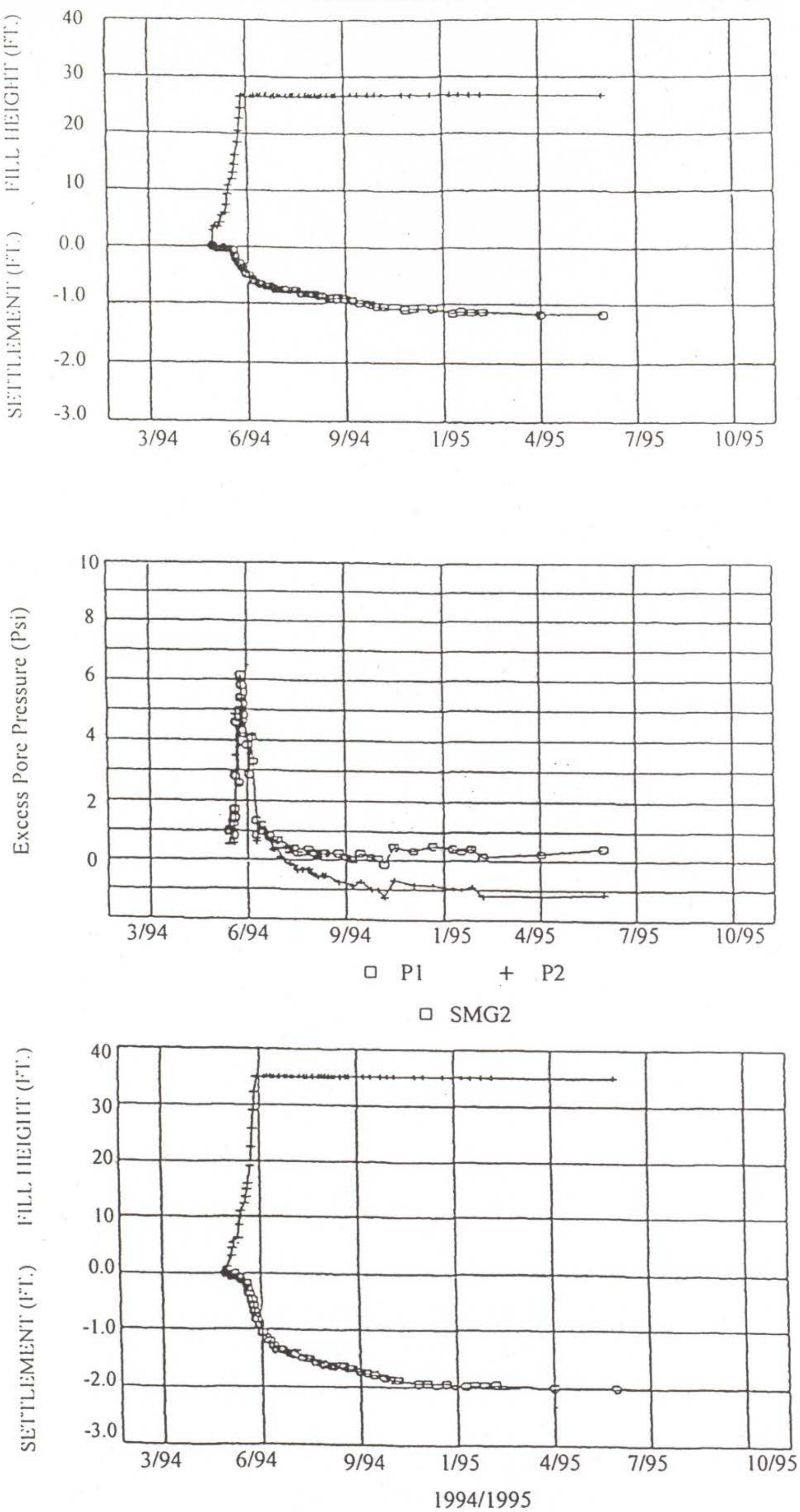


Figure 2-16. Tifft Street west approach embankment field data.

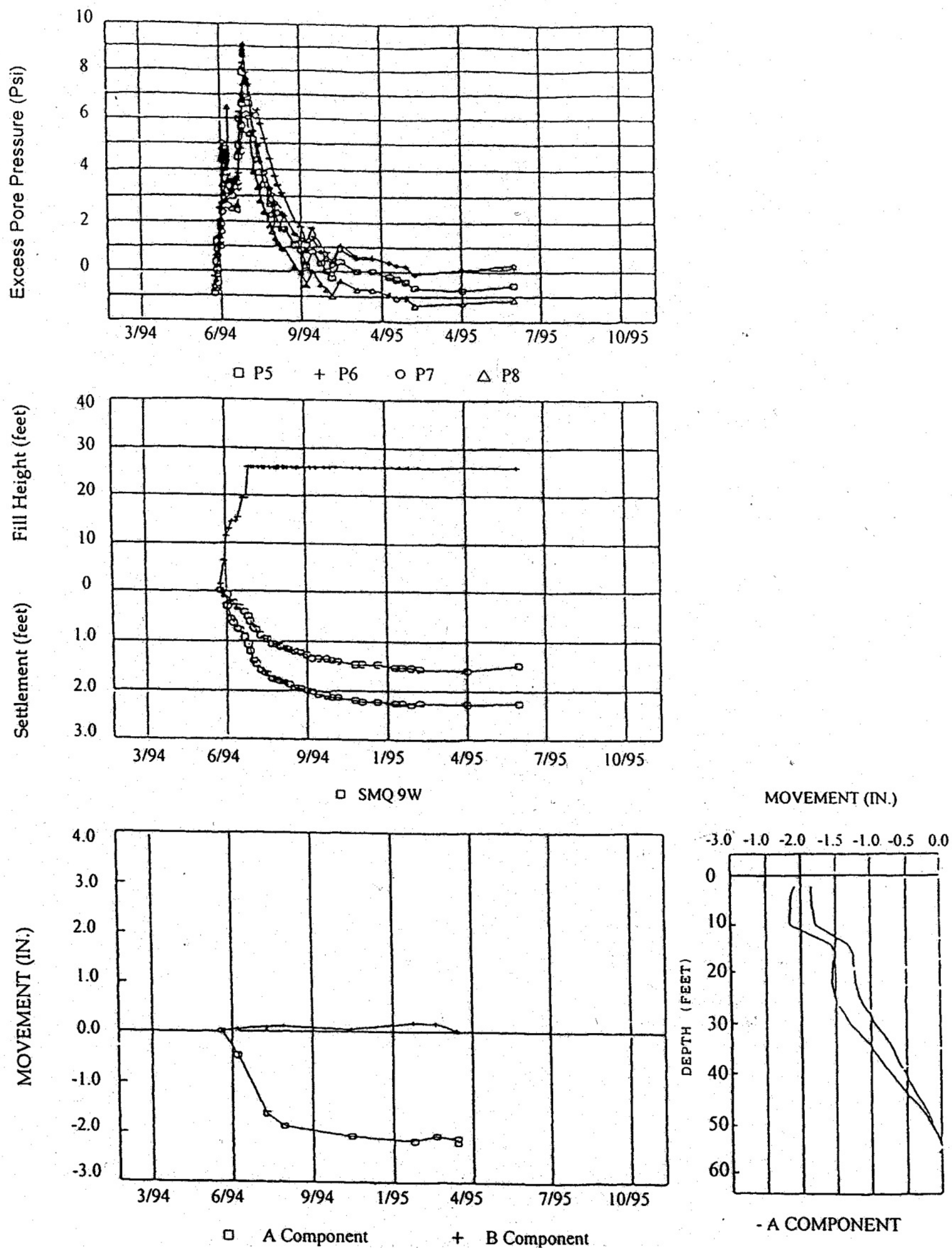


Figure 2-17. Tifft Street east approach embankment field data.

7.3 Jones Creek Substation Site Fill

7.3.1 Project Description

Located in Freeport, Texas the Jones Creek Substation site fill project was a one where the existing ground surface was increased from elevation 5 feet MLS to elevation 23 feet to aid in the creation of an electrical control yard that would be above the potential flood elevation. The project began in July 2015 with site grading. The project scope include clearing and grubbing the site in preparation for grading activities, and installation of PVDs, horizontal strip drains, site fill and surcharge fill. The final grading plan is presented in Figure 2-18.

7.3.2 Subsurface Conditions

The project site is located in southeastern Texas in the gulf coastal plain physiographic province. The area is generally comprised of fluvial and deltaic sediments. The subsoil consists of approximately 2-foot thick topsoil containing loose gray sandy silt (ML). The topsoil is underlain by 23 to 38 feet of soft to stiff gray and reddish brown to brown fat clay with sand (CH), which in turn is underlain by medium dense silty sand of 10 to 15 feet thickness. The groundwater table was generally found to be around 5 feet below ground surface, one day after drilling the bore holes. The fat clay was very soft to medium stiff with blow counts between 3 and 9. The soil has an approximate liquid limit of 69, moisture content of 38%, fines content between 83% and 98%, and a void ratio of 0.95.

7.3.3 Design Concerns

The soft, compressible soils at the sites presented a large risk for long-term potential settlement due the 18 feet of site fill. The site grading was estimated to induce 19 inches of total settlement at the center of the site. Based on the engineers' previous experience with the soils near the project site, they estimated that 10 to 20 percent of the total settlement would occur during the site grading operations (4 to 6 months) and between 25 and 50 percent of the consolidation would occur within the first two years of fill placement. The duration without PVDs and surcharge was too great for the client, due to the sensitivity of the electrical equipment on the planned yard. Therefore, it was deemed necessary to utilize PVDs to decrease the time for consolidation, and the use of surcharge to place the soil into a slightly over-consolidated state. Due the absence of readily available drainage blanket material, the engineer designed for the use of horizontal strip drains with two PVDs attached to each strip drain

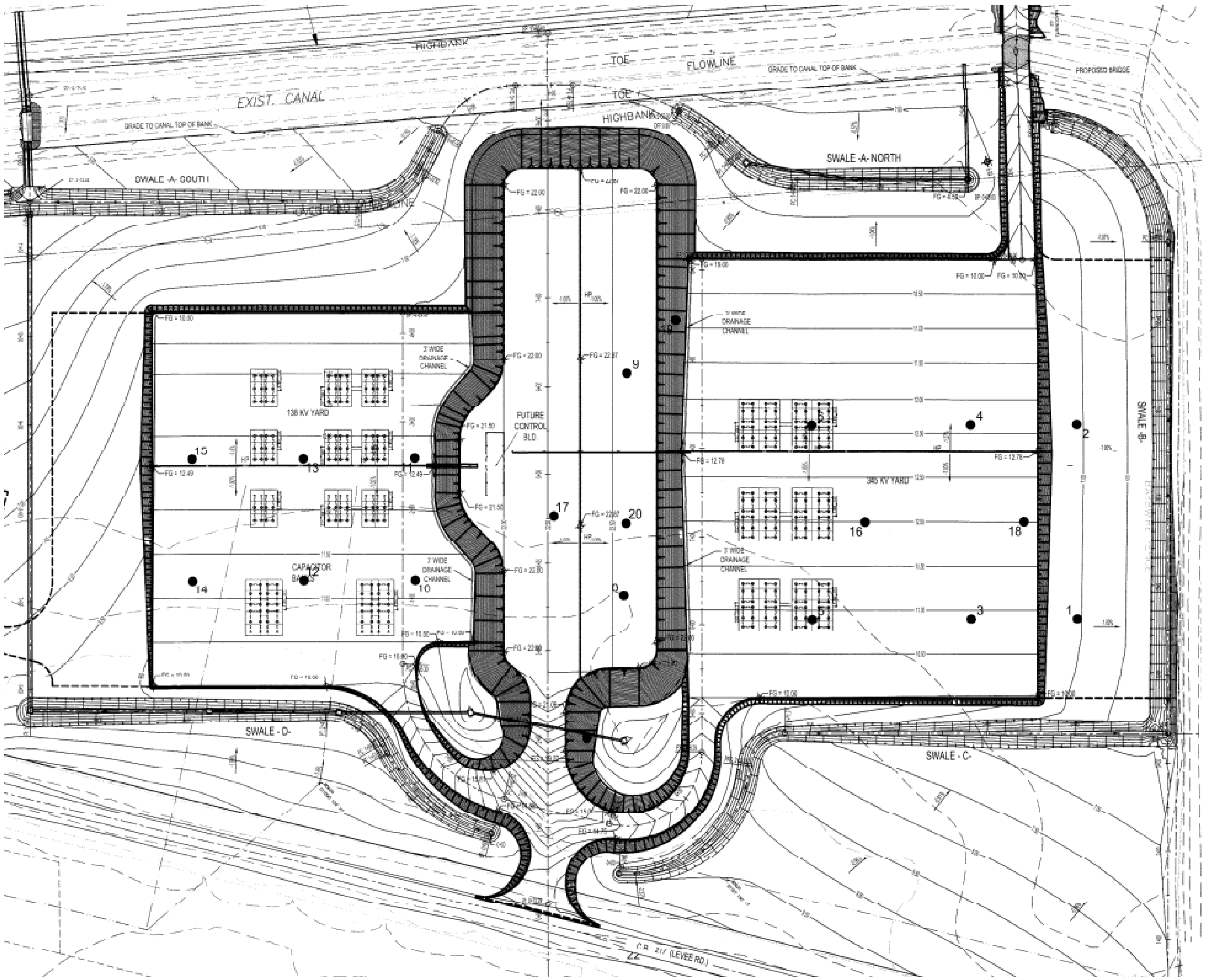


Figure 2-18. Final grading plan for Jones Creek Substation fill project.

7.3.4 Design

Based on the results from three different geotechnical investigations, and subsequent laboratory data, it was determined that a 4-foot triangular spacing would result in 90% consolidation in 4 months following the placement of the fill and surcharge. The design depth for the PVDs was estimated to be an average of 35 feet, with allowance for 2 feet of cut-off allowed to attach the PVD to the horizontal strip drain. Details of the estimated and installed PVDs are listed in Table 2-5.

Table 2-5. Estimated and Installed PVDs

Area	Estimated No. of PVDs	Estimated Length of PVDs incl. Cut-Offs	Estimated Linear Feet of PVDs	Installed Linear Feet	No. of Installed PVDs	No. of Days to Install
1	12,468	37	461,316	446,840	12,468	20

7.3.5 Project Results

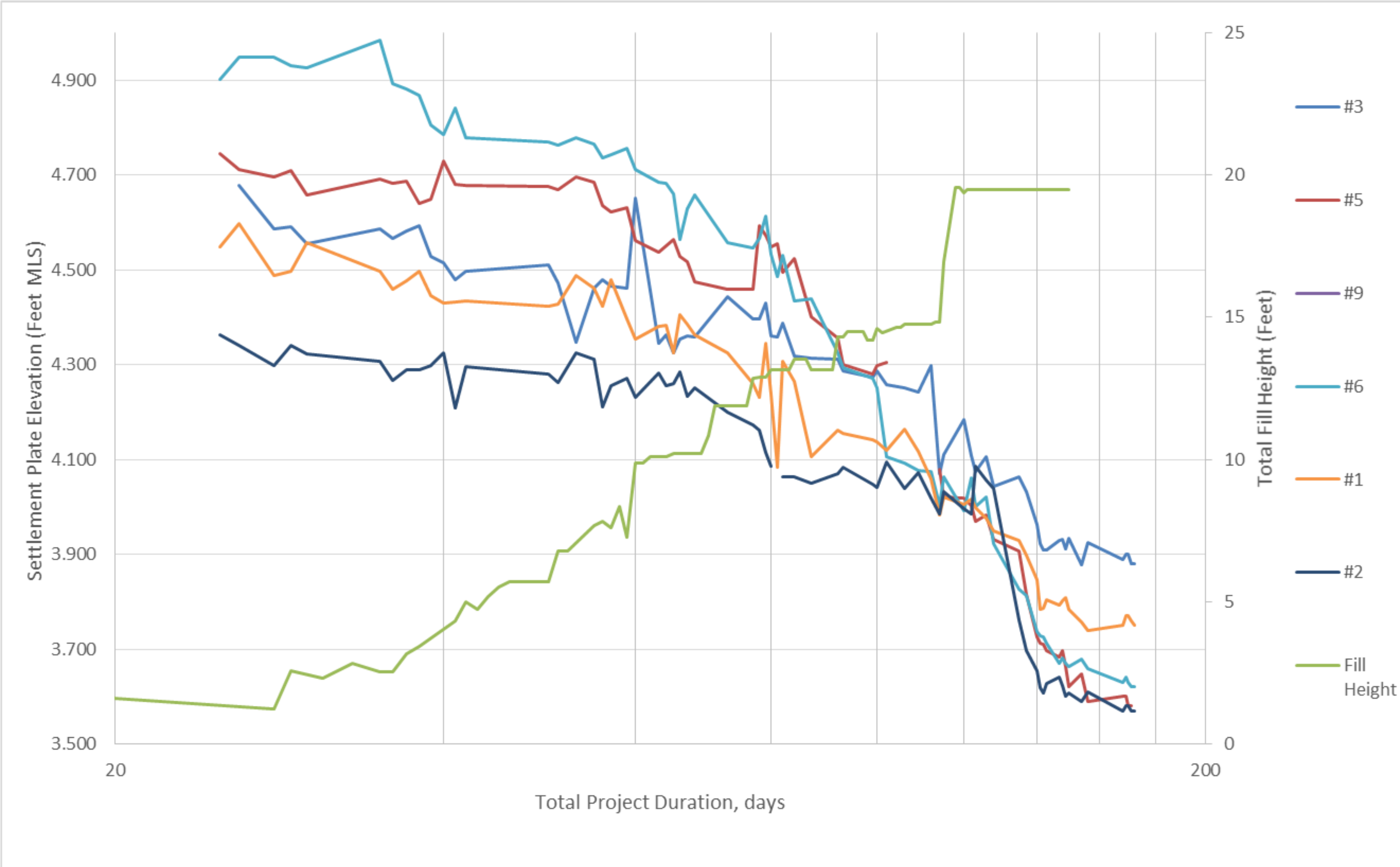
The wick drain material (PVD) used was MebraDrain® 7407, a two piece wick drain with a polypropylene core and seamed geotextile fabric measuring a combined 4.0 inches in width and 3/8 inches thickness. Installation was completed using a hydraulically driven wick stitcher with a 5 inch by 2 inch smooth rectangular mandrel to minimize soil disturbance. The installation of the vertical drain was completed with a three person crew, two for wick drain installation and another for horizontal drain installation. The entire installation period was 20 working days. The total installed quantity of wick drain was slightly less than the estimated value, with an installed average depth of 33.8 feet as opposed to the estimated depth 35 feet.

Total settlement for the project ranged from 8 inches to 15.6 inches, and generally occurred within of 2 months of placement of the fill and surcharge; and was within the design allowance of 4 months. Data recorded via settlement plates is presented in Figure 2-19 against the height of fill and project duration.

7.3.6 Project Cost

There were two wick drain bids for this project comprised of mobilization for a single installation unit, supply and installation of vertical wick drain, and supply and installation of horizontal strip drains. The prices submitted by the winning subcontractor were: $27,500 mobilization, wick drain at approximately $0.45 per linear feet and horizontal strip drain at approximately $1.50 per linear foot.

Acknowledgements **–** Hayward Baker Inc.



Courtesy of Hayward Baker Inc.

Figure 2-19. Settlement plate data and fill height versus project duration.

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Chapter 3

LIGHTWEIGHT FILLS

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1.0 DESCRIPTION AND HISTORY

1.1 Description

The unit density of most compacted soil fills, consisting of sands, silts, or clays, generally ranges from about 115 to 140 pounds force per cubic foot (pcf). The use of such conventional earth fill material can result in significant settlement or decreased stability on some projects. On some projects, it is desirable to use an alternate fill material with a lower unit density to reduce the magnitude of applied earth loads.

In such situations, the use of a lightweight fill material can result in reduced settlement and/or increased stability. The large variety of lightweight fill materials provides a large range in densities, ranging from less than 1 to about 90 pcf. Lightweight fills are most often used to reduce the applied loads to (after Caltrans 2014):

- Eliminate or significantly reduce magnitude of embankment settlement.
- Eliminate or significantly reduce time required to achieve embankment settlement.
- Reduce lateral pressure behind retaining walls, abutments, and other structures.
- Reduce driving force in landslide repair.
- Increase embankment resistance to seismic loads (low unit weight (density) results in lower seismic inertial forces).

1.2 Historical Overview

Many types of lightweight fill materials have been used for roadway embankment construction. Some of the more common lightweight fills are listed in Table 3-1.

Table 3-1. Densities and Specific Gravity for Various Lightweight Fill Materials

Fill Type	Range in Density pcf	Range in Specific Gravity
Geofoam (RCPS)	0.70 to 3.00	0.01 to 0.05
Cellular Concrete	20 to 80	0.4 to 1.3
Wood Fiber	35 to 55	0.6 to 0.9
Tire Shreds	37 to 73	0.6 to 1.2
Expanded Shale, Clay, and Slate (ESCS)	37 to 65	0.6 to 1.0
Fly Ash	70 to 90	1.1 to 1.4
Boiler Slag	60 to 90	1.0 to 1.4
Expanded Air-Cooled Slag	69 to 94	1.1 to 1.5

There is a wide range in the unit weight of these lightweight fill materials, but all have a density less than conventional soils. The composition and sources of the lightweight fill materials listed in Table 3-1 are discussed in Section 3. The availability of some of these materials and, therefore cost, will vary by geographic region.

Lightweight fill materials have been used for decades. The worldwide interest and use of lightweight fill materials led to the publication of a 1997 authoritative reference, *Lightweight Filling Materials*, by the Permanent International Association of Road Congresses (PIARC 1997).

Geofoam is a generic term used to describe expanded polystyrene (EPS) and extruded polystyrene (XPS) material used in geotechnical applications. Geofoam was initially developed for insulation material to prevent frost from penetrating soils. The initial use of geofoam was in Scandinavia and North America in the early 1960s. In 1972, EPS-block geofoam was used as a lightweight fill for a project in Norway (PIARC 1997; Horvath 1995). Today, EPS-block geofoam is widely used by state transportation agencies as lightweight fill for embankment construction; and standards for its use are well established.

The technique of using pumping equipment to inject foaming agents into concrete was developed in the late 1930s. Little is known about the early uses of this product. However, the United States Army Corps of Engineers used foamed concrete as a tunnel lining and annular fill. This product is generally job-produced as cement/water slurry, with preformed foam blended for accurate control and immediate placement. Today, lightweight cellular concrete (a.k.a. foamed concrete) is widely used by state transportation agencies for geotechnical fills.

Wood fiber has been used for many years by timber companies for roadways crossing peat bogs and low-lying land, as well as for landslide repairs (PIARC 1997). The steel-making companies have produced slag by-product since the start of the iron- and steel-making industry. Initially, the slag was stockpiled as waste materials, but beginning around 1950, the slag were crushed, graded, and sold for fill materials (Lewis 1982; National Slag Association 1968; National Slag Association 1988).

Tire shreds are a relatively more recent source of lightweight fill materials. Tire shreds have been used for lightweight fill in the United States and in other countries since the mid 1980s. The availability of this material is increasing each year, and its use as a lightweight fill is further promoted by the need to dispose of tires. Tires are shredded to create tire-derived aggregate (TDA), which can be used as a substitute for gravel, sand and other lightweight fill materials (FHWA 2010).

It is estimated that as of 2013, about 96 percent of scrap tires generated in the United States are used in tire-derived fuel, ground rubber, civil engineering, and other scrap tire markets. Civil engineering applications consist of tire shreds used in roadway construction, landfill construction, septic tank leach fields, and other construction. The civil engineering applications use about 172,000 tons of scrap tires, which is approximately 5 percent of scrap tire use. (https://rma.org/)

Expanded shale, clay, and slate (ESCS) lightweight aggregate has been used for decades to produce aggregate for concrete and masonry units. Beginning in about 1980, lightweight aggregates have also been used for geotechnical purposes. Completed projects include use behind a bulkhead to reduce the lateral pressures on the steel sheeting, for construction of roadways over soft ground, and of the lightweight fill to replace soil in a slope to reduce the gravitational driving force and improves stability and safety of the slope (ESCSI 2007; PIARC 1997; Holm and Valsangkar 1993; Stoll and Holm 1985; Valsankar and Holm 1990).

Waste products from coal burning include fly ash and boiler slag. Both of these materials have been used in roadway construction (PIARC 1997). One of the first documented uses of fly ash in an engineered highway embankment occurred in England in 1950. Trial embankments led to the acceptance of fly ash fills, and other roadway projects were constructed in other European countries. In 1965, a fly ash roadway embankment was constructed in Illinois. Today, the American Coal Ash Association (ACAA) reports that about 2.8 million tons of fly ash is used for embankment and structural fill in the United States per year (ACAA 2014). Boiler slag has been used for backfill since the early 1970s. Approximately 50,000 tons of boiler slag is used for embankment and structural fill in the United States per year (ACAA 2014).

Another byproduct used as lightweight fill is expanded or foamed air-cooled blast furnace slag (ACBFS), from iron and steel manufacturing. A 2009 survey of transportation agencies found little use of lightweight expanded ACBFS in embankment fill applications (Stroup-Gardiner and Wattenberg-Komas 2013c).

1.3 Focus and Scope

The purpose of this technical summary is to present an overview of the more common lightweight fill materials that have been used for geotechnical applications in highway construction. Typical geotechnical engineering parameters for the various lightweight fills that are important for design are provided. In addition, design and construction considerations unique to each of these lightweight fill materials are presented. This information can be used for preliminary planning purposes.

The technical summary discusses preparation of specifications and construction control procedures, and references where detailed information is available at. Approximate costs for the various lightweight fill materials are also presented. Four case histories are presented to demonstrate the effectiveness of lightweight fills for specific situations.

This chapter should not be considered to be a design manual on use of lightweight fill, since the information contained herein is not presented in detail. Furthermore, the engineering properties of some materials can vary significantly depending upon the source of the material and the manufacturing process. The engineering parameters presented in this document are typical values, and a detailed design should include additional testing and evaluation of the contemplated lightweight fill material. References are given so that the reader may obtain more specific information regarding the lightweight fill materials, including properties, performance records, and applications. Key references are listed in Section 1.5.

1.4 Terminology and Acronyms

EPS-block geofoam – generic block-molded product form of expanded polystyrene used in small-strain geofoam applications

Geofoam – block or planar rigid cellular foam polymeric material used in geotechnical engineering applications. (ASTM 2015)

Tire bales are produced by mechanically compressing and tying 100 (automobile) whole tires to form a bale, approximately 2 cubic yards in volume (Zornberg et al. 2005).

Tire chips – pieces of scrap tires that have a basic geometrical shape and are generally between 12 and 50 mm [½ and 2 inches] in size and have most of the wire removed. (ASTM 2012)

Tire derived aggregate (TDA) – pieces of scrap tires that have a basic geometrical shape and are generally between 12 and 305 mm [½ to 12 inches] in size and are intended for use in civil engineering applications. (ASTM 2012)

Tire shreds – pieces of scrap tires that have a basic geometrical shape and are generally between 50 and 305 mm [½ to 12 inches] in size. (ASTM 2012)

Tire shreds are defined by New York State DOT as pieces of scrap tire between 2 inches and 12 inches in size (NYSDOT 2015a).

A number of abbreviations are used throughout this chapter:

- **ACBFS** air-cooled blast furnace slag

- **BFS** blast furnace slag
- **EPS** expanded polystyrene geofoam
- **RCPS** letter designation for rigid cellular polystyrene geofoam covered by ASTM D6817 (2015), includes both EPS and XPS
- **TDA** tire derived aggregate
- **XPS** extruded polystyrene geofoam

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2.0 FEASIBILITY CONSIDERATIONS

2.1 Functions and Design Considerations

Lightweight fill materials have many potential highway applications, principally where the capacity of the underlying soft soil is too low to carry the design load and/or the estimated settlement of the embankment is too large. Under these conditions, the foundation soils must be improved or bypassed, or the embankment load reduced. Lightweight fills fulfill the latter function. A brief overview of lightweight fill use or function is summarized as follows.

2.1.1 Load Reduction

When a fill is placed on soft ground, the main driving force is from the weight of the embankment itself. Conventional methods of improving the foundation stability have included the following (Holtz 1989):

- Removing the soft soil and replacing it with compacted select fill
- Improving the strength of the weak foundation soils through incremental surcharging either with or without vertical drains, to speed consolidation and the development of the shear strength
- Use of deep foundations to transfer some of the load through the weak foundation deposits
- Use of stabilizing berms adjacent to the embankment or flatter side slopes
- Use of some form of ground modification, such as stone columns or soil mixing, to reinforce the weak foundation soils

All of the above solutions require extra time, cost, or the acquisition of additional right of way to allow for construction of the fill. Some of these solutions also require specialized equipment and experienced labor. In these situations, it may be more advantageous to use a lightweight fill material so as to reduce the driving forces, thereby increasing the overall global stability of the fill. The reduction in driving force will depend upon the type of lightweight fill material used. The geotechnical properties of various types of lightweight fill materials are discussed in more detail in Section 4.

A secondary benefit of the use of lightweight fill material is the reduction in settlement under loading. The amount of settlement will be reduced proportionately to the reduction in load.

2.1.2 Shear Strength

Granular lightweight fills have an angle of shearing resistance similar to natural soils, while cellular lightweight fills are characterized by a compressive strength. These properties result in internal stability within the lightweight fills. In the case of an embankment over a weak foundation, the shearing surface will penetrate through the lightweight fill, and the shear strength developed within the lightweight fill deposits will tend to increase the overall global stability.

2.1.3 Compressibility

Many lightweight fill materials, such as cellular concrete, ESCS, fly ash, boiler slag, and air-cooled slag have a compressibility and elasticity similar to natural soils or rock. Under static loading, the amount of internal compression within the fill will often be similar to that for conventional earth fill materials. Under dynamic loading, the resiliency of the lightweight materials will often be similar to the natural soils.

For certain lightweight fill materials, such as shredded tires or wood fibers, the compressibility of these materials under load is significantly larger than compacted soils, and this must be taken into account in the design. Furthermore, the resiliency of these materials is much larger than conventional soils. This property requires a cover of at least 3 feet of soil to be placed over the top of these deposits so as to reduce the resiliency, thereby enabling conventional pavements to be built on these deposits.

EPS-block geofoam compressibility or stress strain behavior is generally linear to stress levels of about 0.5 percent. Beyond that, yielding occurs and the material is subject to time-dependent creep.

2.1.4 Lateral Pressures

The lateral earth pressure at any depth is a function of the vertical overburden pressure multiplied by the coefficient of earth pressure and then reduced by the cohesion of the deposit. In the case of lightweight fills such as cellular concrete or EPS-block geofoam, the cohesion of the material is high and the densities are relatively low. Both of these factors tend to greatly reduce the amount of lateral earth pressure that is transmitted to adjacent structures such, as retaining walls, tunnels, or pile foundations below bridge abutments.

2.1.5 Drainage Characteristics

Many of the granular lightweight fill materials have excellent drainage characteristics. This is favorable for pavement design because the presence of free water in the subgrade results in the following:

- High pore water pressure under traffic loading, which tends to weaken the soil
- Pumping at joints in reinforced concrete pavements
- Plastic deformation of the subgrade soils, thereby leading to rutting
- Heaving of the pavement in areas where frost penetrates into the subgrade

Good drainage is also beneficial behind a retaining wall to eliminate hydrostatic pressures.

2.1.6 Construction in Adverse Weather

It is difficult, if not impossible, to place and compact conventional soils during extremely cold or wet weather. However, some lightweight fills, such as EPS-block geofoam, ESCS, and low-density cellular concrete, have been successfully installed in poor weather.

2.1.7 Seismic Considerations

In Japan, there have been case histories where a highway embankment constructed of geofoam did not fail in a severe earthquake, even though adjacent sections of a soil embankment did (Expanded Polystyrene Development Organization 1994). The lower unit weight of the material results in lower inertial forces under seismic loading.

2.2 Advantages and Potential Disadvantages

All lightweight fill materials have limits on their applicability. The advantages and potential disadvantages of all lightweight fill materials, and of individual materials as applicable, are listed and discussed below.

2.2.1 Advantages

Lightweight fills may be used in several different applications and to perform a variety of functions, as discussed above. Thus, the primary advantages of using lightweight fills may include one or more of the following:

- Accelerate construction
- Reduce structural resistance requirements for lateral loads

- Reduce structural resistance requirements for vertical loads
- Reduce magnitude of and time for consolidation of soft soils and settlement of embankments and structures
- Eliminate or reduce stability problems of embankments and structures

2.2.2 Potential Disadvantages

There are certain limitations to the use of lightweight fill materials. These limitations can be overcome with proper evaluation, design, and construction techniques. The limitations include the following:

- Availability of the materials. Certain geographic areas may have an abundance of one type of lightweight fill material, but not of another. As an example, wood fiber fill would be available in lumber producing areas, fly ash and slag in heavily industrialized areas, and ESCS in areas where production plants are present. Unless the lightweight fill material is available on a local basis, the transportation costs could raise the price considerably, and make these materials non-competitive.
- Construction methods. In general, all lightweight fill materials involve some special procedures with regard to handling, transportation, placement and compaction. Some lightweight fill materials could be difficult to place and handle. Fly ash behaves as silt. When wet, it is very spongy, and when dry it will require dust control. Tire shreds are very resilient when placed, thereby requiring somewhat unconventional compaction procedures. Lightweight cellular concrete requires the use of specialized equipment at the site to introduce air and other additives into the mixture before placement.
- Durability of the fill deposits. Some lightweight fill materials (e.g., EPS-block geofoam) must be protected to ensure longevity. Because EPS-block geofoam is subject to deterioration from hydrocarbon spills, a concrete slab or a geomembrane is generally placed over the surface of the blocks. Wood fibers can decay over a long period of time, although recent studies indicate that the deterioration is limited to the outer surface of an embankment. Fly ash deposits need to be protected with a soil surface to minimize or prevent erosion of the side slopes.
- Wood fiber fills will undergo creep settlement for several years (WSDOT 2015), and distress in and of maintenance of overlying pavement should be anticipated.
- Environmental concerns. Some of the lightweight fill materials may generate leachate as water passes through these deposits. Fortunately, design methods and limitations on applicability have been developed to minimize the amount of leachate, and, to

date, these measures have worked satisfactorily. However, the additional cost of these measures should be considered during design.

- Geothermal properties. Most lightweight fill materials possess geothermal properties that are different than soil. This can lead to accelerated deterioration of flexible pavements and/or problems with differential icing of pavement surfaces due to an alteration of the heat balance at the earth's surface. Essentially, most lightweight fill materials act as thermal insulation, even though this is not an intended or desirable function. However, this can be effectively controlled by placing a suitable thickness (20 to 36 inches minimum) of soil and/or paving materials over the surface of the lightweight fill material.

2.3 Alternative Solutions

The feasibility of using a particular lightweight fill for a project need depends upon the function(s) of the modification and the method(s) selected to carry out the function. Feasibility evaluation includes the identification and evaluation of: technical issues, project development/delivery methods, performance criteria and quality assurance procedures, and non-technical issues that affect the utilization of ground improvement and geoconstruction technologies. A generalized summary of the evaluation process for use of ground modification technologies and materials was presented in Chapter 1.

3.0 LIGHTWEIGHT FILL MATERIALS

3.1 Introduction

Lightweight fill materials include both manufactured and recycled materials. Manufactured materials consist of products such as EPS-block geofoam that are specifically developed or manufactured for special design situations. These materials generally cost more than the recycled materials, but have unique properties that satisfy certain needs.

Recycled materials are generally the product left over from some industrial or commercial process. In the past, some of these materials were considered waste products and were landfilled or stockpiled. It has been found through experimentation that many of these materials have properties, such as low density, low unit cost, or relatively high permeability, that are desirable for fill construction. The use of these materials in roadway embankments has increased during the past 30 years.

For design and construction purposes, lightweight fill materials can be grouped into two broad categories: i.e., materials that have an inherent compressive strength and behave similarly to cohesive soils in undrained loading, and materials that behave and have properties similar to granular soils.

3.2 Lightweight Fill Materials with Compressive Strength

3.2.1 Geofoam

Geofoam is the generic name for any cellular material used in geotechnical applications. The most common geofoam materials used as a lightweight fill, are block molded expanded polystyrene (EPS) and extruded polystyrene (XPS), both plant manufactured. EPS-block geofoam is widely used by state transportation agencies.

The manufacturing process for EPS consists of expanding small beads of polystyrene into spheres that contain numerous closed cells. The expanded spheres are then fused into blocks in a heated vacuum chamber. The heat welds the expanded beads together to form a very light material with a high void content. The final product has an extremely low density, but a relatively high strength and stiffness. The EPS blocks are similar to a large brick, but with typical dimensional ranges of 4 to 16 feet long, 12 to 48 inches wide and 24 to 48 inches thick. Different block size can be made depending upon the size of the manufacture’s mold. Blocks can be factory- or field-cut to any size or shape.

Detailed design guidelines and recommended standards for the use of EPS-block geofoam in embankments and bridge approach fills on soft ground are presented in NCHRP Report 529

(Stark et al. 2004a). The companion document, NCHRP Web Document 65 (Stark et al. 2004b) provides a comprehensive treatment on the use EPS-block geofoam in embankments and includes design examples, construction practices, design details, case histories, and economic analysis. A summary of this NCHRP work on EPS-block geofoam was published by the Transportation Research Board (TRB 2013).

Construction of an EPS-block geofoam embankment is shown in Figure 3-1.

Figure 3-1. Embankment construction with EPS-block geofoam.

3.2.2 Lightweight Cellular Concrete

Lightweight cellular concrete is generally produced by introducing preformed foam (similar in appearance to shaving cream) into cement water slurry. The preformed foam is especially designed for concrete, and creates a network of discrete air cells within the cement matrix. Fly ash may be used as a partial cement replacement in the mixture. The cement water slurry is job-site produced, or thoroughly mixed at a plant and transferred in a ready-mix truck. The mixture may include some sand.

After these materials, (cement, water, preformed foam), are blended to the specified density and thoroughly mixed, the resulting slurry is pumped into place. The lightweight cellular concrete is generally cast in lifts ranging from 1 to 4 feet, or more, depending upon the specific application. Since it is cementitious in nature, hydration solidifies it, and vibratory compaction is not required. Subsequent lifts generally can be placed after a minimum of 12 hours of curing.

The quality of low-density cellular concrete is monitored through its cast density, starting with the wet cast density. The compressive strength is directly related to the cast density of the mixture.

Typically, specialized firms that supply their proprietary foaming agents will blend all materials on the job site and pump it to the required location. The foamed concrete can be produced with only one experienced person and the mixing equipment at the site, using a cement and water mixture delivered by a concrete supply company.

Lightweight cellular concrete backfill of an MSE wall (Chapter 10) is shown in Figure 3-2.

Photo courtesy of MixOnSite USA, Inc.

Figure 3-2. Foamed concrete MSE wall backfill for light rail project in California.

3.3 Granular Lightweight Fills

3.3.1 Tire Shreds

Tire shreds (a.k.a. shredded tires and tire derived aggregate (TDA)) are produced by mechanically cutting tires into chips that are generally in the size range of 4 to 8 inches. These chips are durable, coarse grained, free draining, and have a low compacted density. Each cubic yard of tire shred fill contains about 75 passenger car size waste tires, so there is a potential for using a large number of tires in highway construction. Both steel- and glass-belted tires are used to produce tire shred aggregate.

An application of tire shred embankment fill is illustrated in Figure 3-3.

Photo courtesy of Gale-Tec Engineering

Figure 3-3. Embankment fill construction with tire shreds in Blue Earth County, Minnesota.

Here, a soil fill embankment that was under construction started moving, and threatening the rail line below it, due to underlying soft clay and peat deposits. Lightweight tire shred fill, wrapped in a geotextile, was used to replace soil fill and achieve a stable embankment (Gale et al. 2013).

Historically, the research and development of engineering criteria for use of tire shred lightweight fill was led by Professor Dana Humphrey and colleagues at the University of Maine in the 1990s. In an early FHWA (1997) User Guideline, it was noted that at least 15 states have used tire shreds as lightweight fill and that more than 70 successful projects had been completed on state local or private roads (Humphrey 1996). Some projects used just the tire shreds as embankment fill and some used tire chips blended with soil. The 1997 guideline also noted unresolved issues of: (i) causation of exothermic reactions that resulted in three scrap tire embankment fires in 1995; (ii) optimum particle size and shape of tire shreds; (iii) further investigation into the engineering properties of blended soil and tire shred fills; and (iv) field quality control methods for tire shred embankment construction. A recent survey of states (Stroup-Gardiner and Wattenberg-Komas 2013a), with 45 agencies reporting, reports that 14 states are currently using tire shreds for embankment construction.

The engineering properties of tire shreds have been reported in technical publications and have been summarized by Humphrey (1998). The use of tire shreds for transportation embankment construction has been evaluated by Washington DOT (Baker et al. 2003) and by Texas Tech University (Sonti et al. 2003) for the Texas DOT. These evaluations included lesson learned from previous embankment construction, including some instances of spontaneous combustion of tire shred fills. The NCHRP 435 synthesis (Stroup-Gardiner and Wattenberg-Komas, 2013d), summarizes physical, chemical, environmental, and engineering properties of scrap tire byproducts; and various applications including embankment fill. A recent CalRecycle publication (Cheng 2016) provides application summaries and construction guidelines for TDA fills.

The state of practice for embankment construction with tire shred aggregate is presented in ASTM D6270 (2012) *Standard Practice for Use of Scrap Tires in Civil Engineering Applications*. Today, tire shred embankment fills generally use ASTM D6270 Type A gradation and are:

- limited to a 10-foot maximum thickness,
- encased in a geotextile,
- covered with soil, and
- used above the groundwater table.

Additional design and construction guidance is provided in the New York State DOT GEM-20 manual (2015a). The issues noted in the 1997 FHWA guideline are addressed within the ASTM standard and in the GEM-20 manual, and in the recent reports noted above. Additional discussion on the use of tire shred fills beneath the groundwater table are presented in ASTM D6270, NCHRP 435 (Stroup-Gardiner and Wattenberg-Komas 2013d), and in a MN DOT reports (Oman et al. 2013; Edstrom et al. 2008).

Since the tires are a waste product, the primary costs in some locations are associated with the shredding process and the transportation to the project site. In many locations, the product is used as a fuel source. In some states, there is rebate incentive for using scrap tires. Thus, the economics will depend upon the location.

Scrap tires in two other configurations are also used for embankment construction, though not as frequently as tire shred fills. Tire shreds can be mixed with soil to create a composite fill for embankment construction. Information on soil-tire chip blended fills can be found in the Texas Tech University (Sonti et al. 2003) and the NCHRP 435 synthesis (Stroup-Gardiner and Wattenberg-Komas 2013d).

Scrap tires can also be bound together to create tire bales. A portable baling machine can compresses approximately 100 passenger car sized tires into a bale, bound with galvanized or stainless steel wire, which is about 2 cubic yards in volume and weighing about 1 ton. Thus, the bale has a unit weight of about 40 pcf. See the Colorado DOT Research report (Zornberg et al. 2005) for a summary on use of tire bales, including design, specification, cost, and performance monitoring discussions.

3.3.2 Wood Fiber

Wood fiber includes any type of wood waste generated from the handling of logs at a sawmill. This includes hog fuel, sawdust, and planer chips. Hog fuel is primarily bark and some pulverized wood that has been stripped from logs at a pulp mill. Sawdust is the small size pieces of wood that result from sawing of lumber. The excess material removed when lumber is sawed to prescribed sizes is called planer chips.

The availability of wood fiber for fill construction is geographically limited to locations where a significant timber industry exists. A description of the type of wood fiber used as lightweight fill can be found in reports by Washington DOT (WSDOT) (Kilian and Ferry1993; Allen and Kilian 1993). Placement of wood fiber in embankment construction is illustrated in Figure 3-4.

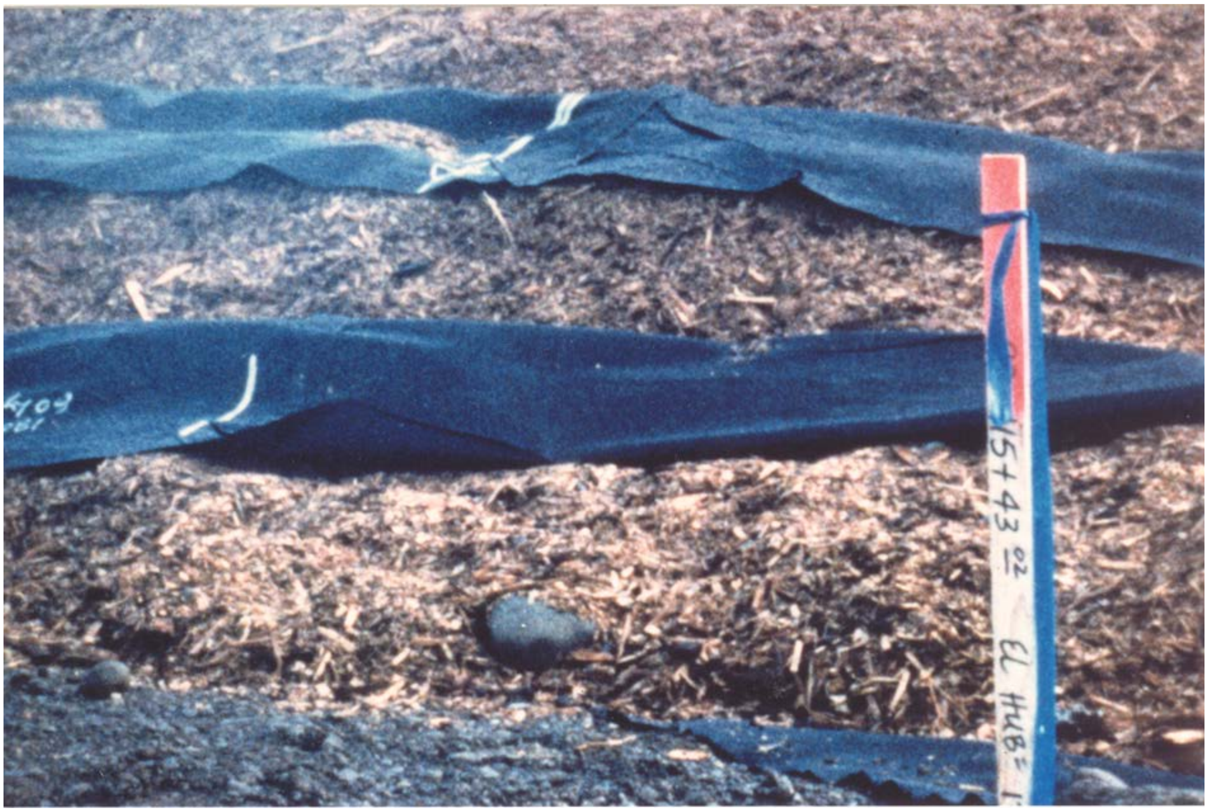


Figure 3-4. Embankment construction with wood fiber.

A summary of WSDOT use of wood fiber fill since the first construction in 1972 is presented in Baker et al. (2003). Over 20 projects were identified. The majority of applications for the lightweight wood fiber fill were landslide corrections, and six applications was embankment construction over soft soils. The average age of the projects reviewed is about 26 years. WSDOT notes it will continue using wood fiber as a primary source of geotechnical lightweight fill.

3.3.3 Expanded Shale, Clay, and Slate (ESCS)

Synthetic aggregates can be produced from shale, clays or slate by heating these materials in a rotary kiln to temperatures in the range of 1,000 to 1,200 degrees C. In this process, the clay minerals of montmorillonite, illite, and kaolinite become completely dehydrated and will not re-hydrate under atmospheric conditions. The expanded vitrified mass is then screened to produce the desired gradation for a particular application. The particle shape may be rounded, cubical, or sub-angular in shape. This aggregate may have a hard ceramic outer shell with a system of pores that vary in size and are largely non-connected. The particles are durable, chemically inert, and relatively insensitive to moisture (will absorb and retain some water).

The cost for producing ESCS materials is relatively high (though comparable in cost to other manufactured lightweight fill materials). For this reason, these products have generally been used as lightweight aggregates for structural concrete, but are used for some embankment construction. However, in areas where high-quality, naturally occurring aggregates are no longer present, the ESCS have been used to produce synthetic aggregates for normal roadway construction.

The placement of expanded clay lightweight fill during the early stages of embankment construction is shown in Figure 3-5.

Figure 3-5. Placing expanded clay fill.

3.3.4 Fly Ash

When coal is burned, the finer portion of the residue is airborne and carried to the smoke stack where it is collected via a precipitator. This portion of the waste product is classified as fly ash. Fly ash is defined as *the finely divided residue that results from the combustion of ground or powdered coal and is transported from the combustion chamber by exhaust gases*. Fly ash consists of very fine particles that are predominantly in the silt-sized gradation range. The particles are rounded and consist of a siliceous material.

The chemical properties of fly ash depend upon the type of coal that is burned. ASTM standard C618 defines two categories of fly ash. Class C fly ash is produced from lignite or sub-bituminous coal, and has pozzolanic and some cementitious properties. Some Class C fly ashes may contain lime contents higher than 10 percent.

Class F fly ash is produced from burning anthracite or some bituminous coals that meet the applicable requirements given in ASTM C618. This class of fly ash has pozzolanic properties.

In recent years, the primary use for Class C and Class F fly ashes has been as an ingredient in concrete to reduce the cement content. This demand has raised the cost of the fly ash to the level where it is less competitive with other lightweight fill materials. Fly ash is sometimes a partial cement replacement in lightweight cellular concrete.

3.3.5 Blast Furnace Slag

Blast furnace slag is generated from the production of iron. In the process, the blast furnace is charged with iron ore, limestone and/or dolomite flux, and coke for fuel. Two products are obtained from the furnace: molten iron and slag. The slag consists primarily of the silica and alumina from the original iron ore combined with the calcium and magnesium oxides from the flux stone. The slag leaves the furnace in a form of liquid resembling lava. Depending upon the manner in which the molten slag is cooled and solidified, three distinct types of blast furnace slag can be produced:

- Air-cooled slag. The molten slag is permitted to run into a pit. Solidification takes place under the prevailing atmospheric conditions, after which cooling may be accelerated by water spray. After the pit has been filled and cooled, the slag is dug, crushed, and screened to the desired aggregate sizes. The resulting slag aggregate is angular and vesicular. The uniformly graded slag has the lowest density, which is desirable for use as a lightweight fill material.
- Expanded slag. Treatment of the molten slag with controlled quantities of water accelerates the solidification and increases the cellular nature of the slag, producing a lightweight product. Either a machine or pit process may be used to mix the water and molten slag.
- Granulated slag. Molten slag that is quickly chilled will form a glassy granular product called granulated slag. This process is the most rapid of the cooling methods, and little or no crystallization occurs. The granulated slag may be crushed and screened or pulverized for various applications.

Air-cooled blast furnace slag is the predominant form of slag processed in the United States, accounting for 90 percent of the blast furnace slag sales in 1992 (Solomon, 1993). The expanded slag and granulated slag are of lower unit weight and are more expensive to produce. Their use is generally limited to aggregates for lightweight concrete or concrete block.

The principal constituents of blast furnace slag are the oxides of silica, alumina, lime, and magnesia. These oxides comprise 95 percent or more of the total. The remaining portion of the slag contains manganese, iron, and sulfur compounds. Certain slag containing high levels of soluble sulfur can be very acidic and, therefore, corrosive.

Steel slag is another slag produced from the steel-making operations. Steel slag is undesirable for mass filling, such as embankments, because it contains the oxides of calcium and magnesium, which are expansive. This expansive nature has resulted in heave of concrete slabs. In the past, many slag piles consisted of a mixture of both blast furnace and steel slag, but after the undesirable effects of expansion from the steel slag became known, the industry began to separate slag piles so that the blast furnace slag could be processed for roadway construction. Steel slag is used in asphaltic concrete road surfacing mixes.

3.3.6 Boiler Slag

Boiler slag is a coal combustion byproduct from power plants. It is obtained from molten ash collected in wet, water cooled bottom boilers. It has been estimated that approximately 25 percent of all power plant ash produced in the United States is boiler slag, 65 percent is fly ash, and 10 percent is dry bottom ash and cinders. Dry bottom ash is highly absorptive and not suitable for most embankment fills. However, boiler slag is a durable aggregate. It is formed when the slag flows from the furnace in a hot molten condition and is discharged in cold water where it crystallizes, solidifies, and forms angular black glassy particles usually less than ⅛-inch to ⅜-inch in size. It is generally composed of silica and ferric oxide particles of angular and irregular shape.

4.0 DESIGN CONCEPTS

4.1 Design Considerations

The function of lightweight fills is to reduce the load on the foundation. A number of factors must be evaluated in order to determine appropriate candidate(s) fills. These include:

- Availability of lightweight fill materials in the geographic area of the project.
- The geotechnical engineering properties of the lightweight fill material for use in the design evaluation. The same, or similar, engineering analyses used for conventional fills would be used to evaluate the suitability of the design using granular lightweight fill materials. For example, if slope stability is a concern, the reduced density of the lightweight fill plus the angle of shearing resistance or cohesion of the lightweight fill should be included into the slope stability calculations (Holtz 1989; Stark et al. 2004a). If settlement is a concern, the reduced density of the fill should be used in the settlement calculations. For fills characterized by compressive strength (EPS-block geofoam, lightweight cellular concrete), some unique design criteria must be evaluated (Stark et al. 2004a).
- Drainage requirements and drainage characteristics of lightweight fill materials.
- An evaluation of the durability, compressibility, water absorption potential, degradation potential, and other unique characteristics of the lightweight fill materials.
- Design and construction considerations. It may be necessary to incorporate certain features into the design to compensate for potential environmental problems or to reduce erosion potential. Construction with lightweight fill materials may require that certain field procedures be followed.
- An evaluation of the costs for using lightweight fill versus conventional construction. Costs for various types of lightweight fill projects are discussed in Section 7. However, the cost will vary for each particular site depending upon the availability of the lightweight fill materials at the project site.
- An evaluation of the costs for the various lightweight fill materials. Costs generally increase as unit weight of materials decrease, so the design needs should be balanced with the design requirements.

4.2 Design Procedures

As a guide for preliminary planning and design, Tables 3-2 through 3-10 summarize some of the important geotechnical engineering parameters for lightweight fill materials.

Environmental, construction, and design considerations are also listed in these tables. Each of these factors should be considered for designs using lightweight fill materials.

The design considerations are guidelines that experience has proven to be appropriate when working with a particular lightweight fill material. Construction guidelines refer to techniques that have been developed in the field to achieve proper densification and to minimize construction problems.

The information presented in these tables should be used as guidelines only. For each specific project site, additional testing should be performed to determine the material design properties of the lightweight fill material that will be used. This is especially true for the recycled materials, since the chemical composition of these materials, as well as the gradation or shape after recycling, could vary from one source to another.

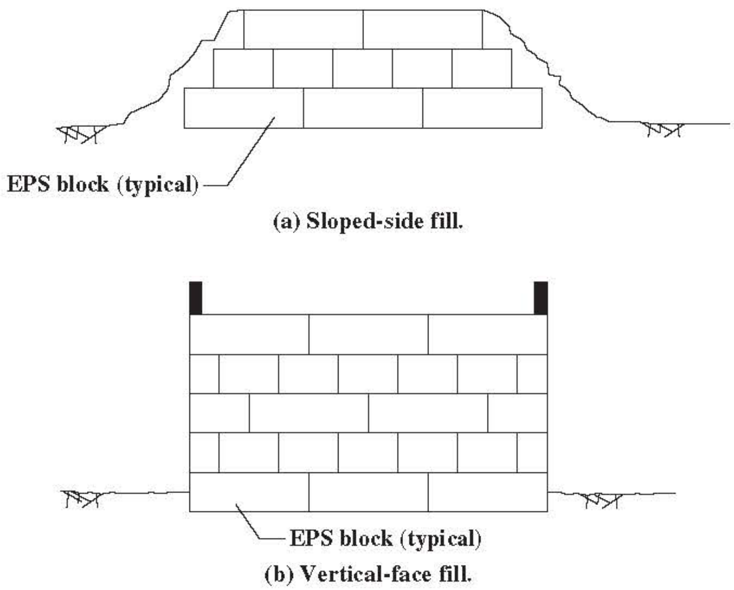
In the case of manufactured lightweight fill materials, the minimum compressive strength or the maximum density of the product can be specified in advance, and the final product is relatively uniform in consistency. In this case, testing would confirm that the desired strength or density has been achieved.

4.2.1 Fills with Compressive Strength

4.2.1.1 EPS-block Geofoam

The most comprehensive design, material, and construction guidelines on the use of EPS-block geofoam for highway construction are presented in NCHRP Report 529 (Stark et al., 2004a), with particular application to highway embankments. Comprehensive guidelines including design examples and design detailing are contained in the report companion document NCHRP Web Document 65 (Stark et al. 2004b). An earlier monogram, by Horvath (1995), provided (then current) comprehensive information on the use of EPS-block geofoam in lightweight fill applications. A subsequent NCHRP study provides guidelines for EPS-block geofoam applications specific to slope stability projects (Arellano et al. (2011).

Design of EPS-block geofoam embankments must address both strength and serviceability. The NCHRP Report 529 (Stark et al., 2004a) presents detailed design procedure flowcharts for EPS-block geofoam embankments and design charts for various embankment configurations and design loadings. Typical EPS-block geofoam applications of stand-alone embankments are shown in Figure 3-6.



Horvath 1995, from Stark et al. 2004a

Figure 3-6. Typical EPS-block geofoam applications involving standalone embankments.

The overall design process when using EPS-block geofoam is divided into three phases in order to consider the interaction between the three major components in the embankment (Stark et al. 2004a).

1. Design to preclude external (global) stability of the embankment. This should include considerations for settlement, bearing capacity, and slope stability under the projected loading conditions.
2. Design for internal stability within the embankment mass. The design must ensure that the EPS-block geofoam mass can support the overlying pavement system without excessive immediate and creep compression.
3. Design of an appropriate pavement system for the subgrade provided by the underlying geofoam blocks.

The major components of an EPS-block geofoam slope system are shown in Figure 3-7.

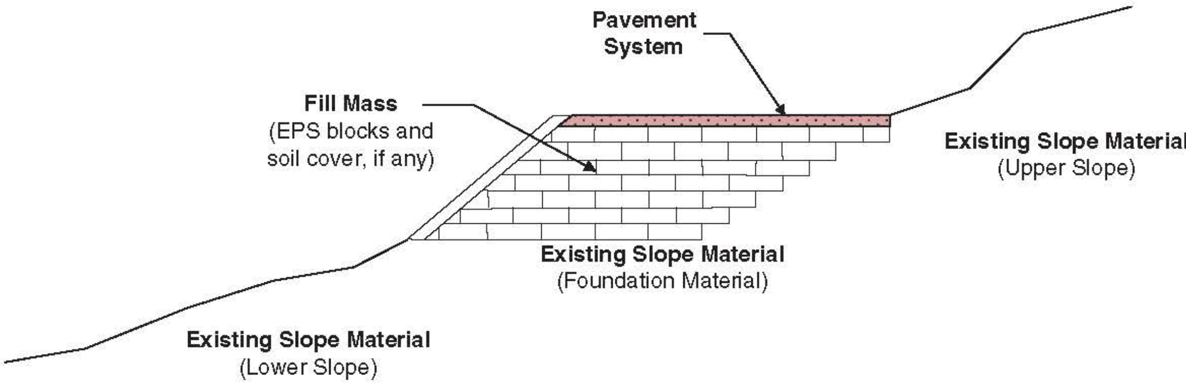


Figure 3-7. Major components of an EPS-block geofoam slope system.

The design of such a system must address the same three failure modes as an EPS-block geofoam embankment system, namely external stability, internal stability and the pavement support system. Detailed design procedure flowcharts for EPS-block geofoam slope systems are provided in Arellano et al. (2011) and NCHRP RRD 380 (2013). A complication in the evaluation of slope systems is that multiple potential slip surfaces must be analyzed including potential slip surfaces above and below the EPS-block geofoam and global stability failure that encompasses the EPS-block geofoam (see Figure 3-8).

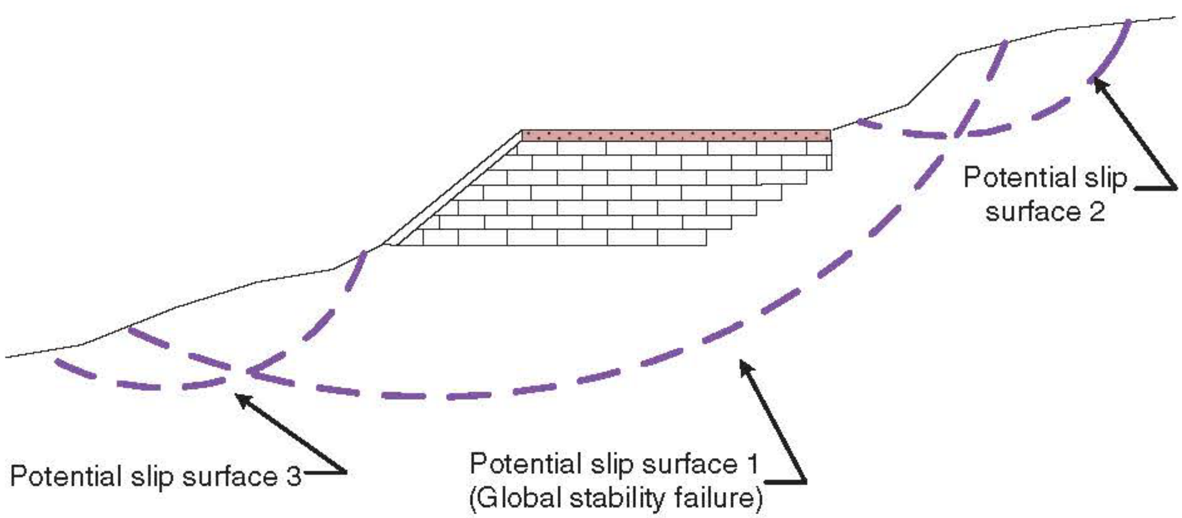


Figure 3-8. Multiple potential failure surfaces to consider with EPS-block geofoam slope systems.

Stability analyses of embankment and slope systems require the modeling and quantifying of both the internal shear strength of the EPS-block geofoam and of the geofoam interfaces. The internal shear strength of EPS-block geofoam correlates to its compressive strength. The interfaces typically include geofoam to geofoam, geofoam to soils, and geofoam to geosynthetic material. Interface friction is an important stability design consideration, particularly under horizontal wind, water and/or seismic loading conditions.

EPS-block geofoam embankments and slope systems often support an overlying roadway pavement. The objective in the design of an appropriate pavement system is to select the most economical arrangement and thickness of pavement materials for the subgrade provided by the supporting EPS blocks. Equivalent soil subgrade strengths for the EPS blocks can be used with traditional pavement design procedures. Subgrade properties as a function of EPS block density are listed in Table 3-2 (Stark et al. 2004a).

Table 3-2. Equivalent Soil Subgrade Values of EPS-Block Geofoam for Pavement Design

EPS Block Density (pcf)	CBR (%)	Young's Modulus (psi)	Resilient Modulus (psi)
1.25	2	725	725
1.5	3	1015	1015
2.0	4	1450	1450

The use of EPS-block geofoam requires the use of materials standards to define material properties. There are two materials standards are currently in use by state departments of transportation from ASTM International and from NCHRP reports.

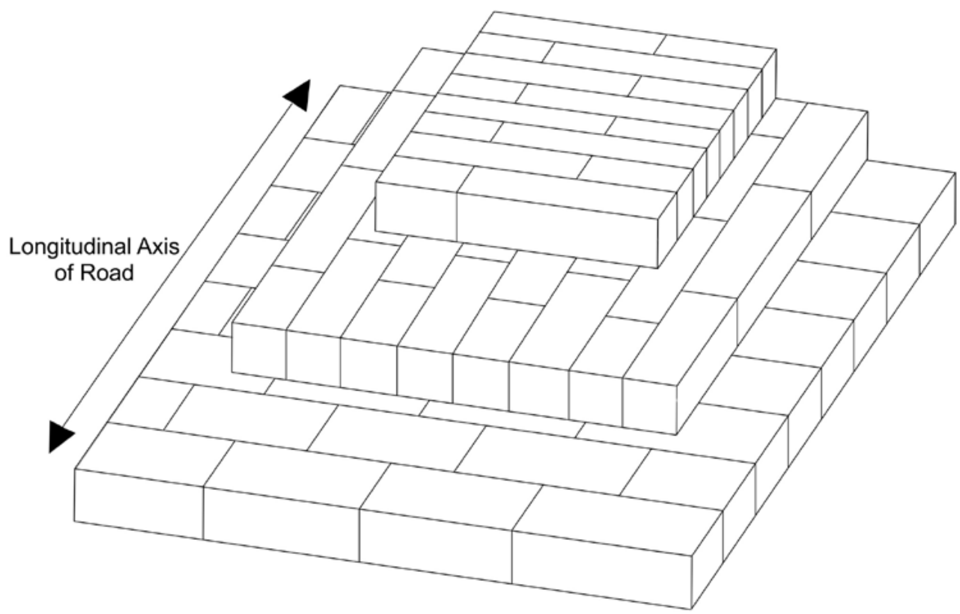
ASTM International has three standards for the use of RCPS geofoam. ASTM D6817 covers physical properties and dimensions, including densities and compressive resistance at 1% strain. There are seven grades of EPS listed in ASTM D6817 (ASTM 2015) that range in density from 0.70 to 2.85 pcf, with compressive resistance values of 2.2 to 18.6 psi, respectively. Six grades of XPS are listed with densities and compressive strengths ranging from 1.20 to 3.00 pcf and 2.9 to 40.6 psi, respectively. ASTM D7180 (ASTM 2013a) covers design considerations for the use of EPS-block geofoam in geotechnical projects. ASTM D7557 (ASTM 2013b) covers quality assurance sampling issues for EPS-block geofoam.

NCHRP Report 529 (Stark et al. 2004a) provided a recommended combined material, product, and construction standard covering EPS-block geofoam for use as lightweight fill in stand-alone road embankments and related bridge approach fills on soft ground. This standard contained recommendations on manufacturing quality assurance, product shipment and storage, construction quality assurance, site preparation, placement of EPS-block geofoam, and pavement construction. The recommended standard was intended to be used to create a project-specific specification. Nichols (2008) noted that the standard had been included in FHWA publications as a recommended design and construction standard. In the follow-on study involving EPS-block geofoam slope systems (Arellano et al. 2011), this standard was updated based on experience in the use of EPS-block geofoam in embankments and extended to include EPS-block geofoam slope systems. Key design and construction considerations for EPS are summarized in Table 3-3.

Table 3-3. Expanded Polystyrene (EPS) Design and Construction Guidelines

Item	Guideline
Design Parameters	**Density, Dry**: 0.75 to 2.0 pcf **Compressive and Flexural Strength**: Varies with density, 6 to14 psi **Modulus of Elasticity**: 580 to 1450 psi **California Bearing Ratio (CBR)**: 2 to 4 **Coefficient of Lateral Earth Pressure**: Lateral pressures from adjacent soil mass may be reduced to a ratio of 0.1 of horizontal to vertical pressure (PIARC 1997).
Environmental Considerations	There are no known environmental concerns. No decay of the material occurs when placed in the ground.
Design Considerations	EPS blocks will absorb water when placed in the ground. Blocks placed below water have resulted in densities of 4.8 to 6.4 pcf after 10 years. Blocks above the water had densities of 1.9 to 3.2 pcf after 10 years. For settlement and stability analyses, use the highest densities to account for water absorption. Buoyancy forces must be considered for blocks situated below the water table. Adequate cover should be provided to result in a minimum safety factor of 1.3 against uplift. Because petroleum products will dissolve geofoam, a geomembrane or a reinforced concrete slab is used to cover the blocks in roadways in case of accidental spills. Differential icing potential of pavement, due to a cooler pavement surface above the EPS versus pavement above a soil only subgrade. Differential icing can be minimized by providing a sufficient thickness of soil between the EPS and top of pavement surface. Use side slopes less than or equal to 2H:1V and a minimum cover thickness of 0.8 feet. If a vertical face is needed, cover exposed face of blocks with shotcrete or other material to provide long-term UV protection.
Construction Considerations	The subsoil should be leveled before placement of geofoam blocks. A layer of sand/gravel is frequently placed as a leveling course. When multiple layers of geofoam blocks are placed, the blocks should be placed at right angles to avoid continuous vertical joints and to promote interlocking. See Figure 3-9. A minimum of two layers of blocks must be used. Provide a mechanical connection between blocks using a galvanized barked plate for shear transfer. Place cover material over geofoam blocks as soon as possible to prevent displacement from wind or buoyancy. Avoid prolonged exposure to sunlight, which embrittles EPS.

The NCHRP combined material, product, and construction standards and the ASTM material standards have been beneficially applied to geofoam projects in roadway fill, slope systems, and other applications. However, the presence of two standards has caused some confusion to users (Nichols 2008). In addition to differences in material designations, the primary difference between the two material standards is the “unit weight to compressive strength” relationships for block designations. There is long-standing industrial acknowledgment that EPS unit weight could be reliably indexed to the elastic limit stress – measured as the compressive stress at 1% strain – of the material. However, there is currently disagreement within the industry as to what elastic limit stress can be practically achieved for a given block unit weight, and what that relationship means with respect to the performance of the blocks in a specific fill application (Nichols 2008). Additional discussion of the use of the two standards in highway applications can be found in Nichols (2008), Arellano et al. (2011), and Horvath (2012).



After Stark et al. 2004b

Figure 3-9. Isometric view of typical EPS block layout.

4.2.1.2 Cellular Lightweight Concrete

Cellular lightweight concrete (a.k.a. foamed concrete) is placed as a liquid product that is practically self-leveling, and can be pumped over a distance as great as 3,300 feet. The cellular lightweight concrete will begin to harden between 2 to 6 hours after placement, and generally solidifies in 24 hours. Design with this product is analogous to design with conventional concrete. The maximum cast unit weight and related minimum compressive strength should be specified as dictated by design and with consideration of local suppliers of cellular lightweight concrete. The range of wet cast density and compressive strength that can be specified generally can range from 20 to 80 pcf and 10 to 300 psi, respectively.

Some additional design considerations for pavement support applications include the following (CROW 2003):

- The heat conductivity of foamed concrete is limited. Therefore, it may not be advisable to construct flexible pavements directly over the product.
- The dynamic Modulus is nearly equal to the static modulus. To account for fatigue under repeated loads, the tensile strength should consider a factor of safety of about 2. Additional tensile capacity can be obtained by the inclusion of steel mesh reinforcement.
- Poisson's ratio for foamed concrete is about 0.20.
- The cost of foamed concrete increases with cast density.

Key design and construction considerations are summarized in Table 3-4.

Table 3-4. Cellular Lightweight Concrete Design and Construction Considerations

Item	Guideline
Design Parameters	**Wet Density Range:** 20 to 80 pcf **Compressive Strength Range:** 10 to 300 psi, depending on density **Water Absorption:**1.4 to 15 psf, depending on density **Freeze-thaw Resistance, 100 Cycles:** 92 to 98%, depending on density **Coefficient of Lateral Earth Pressure:** Negligible for vertical loads applied directly over the foamed concrete. Lateral pressures from adjacent soil mass may be transmitted undiminished.
Environmental Considerations	There are no known environmental concerns.
Design Considerations	Dry density values will be lower than wet density values. Buoyancy could be a problem if foamed concrete is placed below the water table and there is not sufficient vertical confinement. The lower compressive strength mixes are affected by freeze-thaw cycles. The product should be used below the zone of freezing or a higher compressive strength used. Densities greater than 37 pcf have reported excellent freeze-thaw resistance. There is some absorption of water into the voids, which could affect the density and compressive strength. Saturation by water should be prevented by construction of a drainage blanket and drains.
Construction Considerations	A staging area is required for batching, mixing, and placing on site. The foamed concrete is very fluid; formwork should be tight to avoid flow of the mix through joints or gaps in the forms. Polyethylene film may be used to prevent leakage. If the foamed concrete is placed in a confined area, forms are not necessary, as the fluid mix will flow to completely fill the void. The lift thickness should not exceed 4 feet, as the heat of hydration would have an adverse effect on the foam. Allow a minimum 12-hour waiting period between lifts. No special provisions for cold joints are necessary, although each lift surface should be scarified and clean. If shaping is required, the lift thickness should be limited to 2 feet to allow workers to shape the surface while wading in the fluid mix.

4.2.2 Granular Lightweight Fills

The important design and construction criteria associated with granular lightweight fills are summarized in Tables 3-5 through 3-10. Design proceeds using the indicated variables and conventional geotechnical methods associated with granular soils.

Table 3-5. Tire Shreds Design and Construction Guidelines

Item	Guideline
Design Parameters	**Dry Density:** 21 to 53 pcf loose and 30-73 pcf compacted, with various of gradations and reporting sources **Angle of Shearing Resistance:** 19° to 30° **Cohesion Intercept: 100** to 230 psf, use 0 for design **Compressibility:** 5 to 40 percent vertical strain over a range of 200 to 4,200 psf vertical stress **Permeability:** 0.5 to 60 cm/sec **Type A Gradation (ASTM D6270)**: 8-inch maximum dimension; 100% passing 4-inch, a minimum of 95% passing 3-inch, a maximum of 50% passing the 1.5-inch, and a maximum of 5% passing the 0.2-inch sieve **Coefficient of Lateral Earth Pressure:** 0.25 to 0.47
Environmental Considerations	The design considerations listed below address minimizing leachate generation and transport from tire shred fills. See NCHRP 435 (Stroup-Gardiner and Wattenberg-Komas 2013d), ASTM (2012), Minnesota DOT (Edstrom et al. 2008), Washington DOT (Baker et al. 2003) for additional information and discussion on environmental considerations.
Combustion Potential	The tire shred gradation and design considerations were developed, in part, to prevent combustion of tire shred fills. These design details prevent or minimize the amount of infiltration of water and air into tire shred fill.
Design Considerations	Limit layers to 10 feet in thickness. Keep the tire shred fill above the water table. Provide good surface drainage of roadway surface to avoid water seepage through the shredded-tire fill. Tire shreds should be separated from the surrounding soil by completely wrapping with a geotextile. Metal fragments must be firmly attached to the chips, with 98 percent embedded in the rubber to prevent exposed wire strands from puncturing tires or construction equipment. Place a minimum 3-foot thick soil cap on the top and side slopes of the tire chip fill to minimize pavement deflections and provide confinement. Place 2-foot soil surcharge for 60 days to minimize post construction settlement due to compressibility of tire shreds. Top of tire shred embankment should be a minimum of 5 feet below the top of subgrade elevation. Multiple 10-foot tire shred layers should be separated by 3 feet of soil fill. Drainage pipes beneath the fill should be located at least 3 feet below the bottom of the of the tire shred layer. Drainage features that could provide free access to air should be avoided at the bottom of the fill.
Construction Considerations	Spread using a track-mounted dozer in a lift thickness of 3 foot or less. Compact using sheep's foot rollers, smooth drum rollers, or by repeated passes with a D-8 dozer. Use multiple passes of compaction equipment, since compressibility decreases after 5 to 8 cycles of loading. Anticipate 35 percent volume reduction during compaction, plus 10 percent shrinkage under loading of soil cover and pavement base course.

Table 3-6. Wood Fiber Design and Construction Guidelines

Item	Guideline
Design Parameters	**Moist Density:** 45 to 60 pcf **Angle of Shearing Resistance:** Sawdust – 25° to 27° Hogfuel – 31° Wood Chips – 30° to 49° **Permeability:** 1 x 10^{-5}m/s **Compressibility:** Loose volume reduces 40 percent on compaction. **Vertical subgrade reaction coefficient:** 1300 to 1450 psi in top 2 feet, roughly corresponding to a CBR of 1
Environmental Considerations	Potential environmental effects of the leachate include: depletion of available dissolved oxygen in groundwater. lowering of groundwater pH because of acidic nature of leachate, which has pH of 4 to 6. potential contamination of water with toxins. Methods to reduce contamination include: reducing water infiltration into wood fiber by drains and capping. treatment of leachate. barriers between wood fiber fill and adjacent bodies of water.
Design Considerations	Restrict particle size to 6 inches maximum to prevent development of large voids. Less than 30 percent should be finer than 0.5 inches to minimize the use of fine uniform sawdust. Use fresh wood fiber to prolong the life of the fill. Use side slopes of 1.5H:1V or flatter. Employ surface treatment with cover material of thickness 2 feet or more to protect slope from erosion and minimize deterioration of wood fibers. Restrict height of fill to about 16 feet and reduce air penetration into wood to minimize the possibility of spontaneous combustion.
Construction Considerations	Truck-mounted equipment is used to spread fiber in 12 to 20-inch lifts. Two passes with a fully loaded hauling truck weighing 33 kips or more is usually sufficient to properly compact wood fiber.

Table 3-7. Expanded Shale, Clay, and Slate (ESCS) Design and Construction Guidelines

Item	Guideline
Design Parameters	**Dry Density, Compacted:** 50 to 65 pcf **Dry Density, Loose:** 40 to 54 pcf **Angle of Shearing Resistance:** 35° loose, 37° to 44° compacted **Grain Size Gradation:** 3/16 to 1 inch **Permeability:** High **Coefficient of Subgrade Reaction:** 33 to 37 pci loose, 140 to 155 pci compacted
Environmental Considerations	There are no known environmental concerns.
Design Considerations	The material will absorb some water after placement, when continually submerged. Samples compacted at a water content of 8.5 percent have been found after 1 year to have a water content of 28 percent. Over a longer period of time, the estimated long-term water content would be about 34 percent. Side slopes of embankments should be covered with a minimum of 2.5 feet of soil cover. Use side slopes of 1.5H to 1V or flatter to confine the material and provide internal stability. For calculating lateral earth pressures, use an angle of shearing resistance of 35°.
Construction Considerations	Particle degradation can occur from steel-tracked construction equipment. Use 2 to 4 passes with rubber-tired rollers and lift thickness of 3 feet or less. Fill should be unloaded at side of fill area, then distributed with lightweight equipment with a contact pressure of 4.5 psi or less. Field density may be approximated in the laboratory by conducting a modified one-point AASHTO T 272 density test.

Table 3-8. Fly Ash Design and Construction Guidelines

Item	Guideline
Design Parameters	**Density Range, Compacted:** 70 to 90 pcf **Shear Strength:** 33° to 40°, c = 0, for Type F; Class C is self-hardening, so the shear strength will vary as it cures **Permeability:** Range of 1 x 10^{-6} to 1 x 10^{-9} m/s **Compressibility:** C_c = 0.05 to 0.37, C_{cr} = 0.006 to 0.04 **Grain Size Range:** 0.005 to 0.074 mm **Specific Gravity:** 1.9 to 2.5 **Atterberg Limits:** Non-plastic
Environmental Considerations	The leachate is alkaline, with pH of 6.2 to 11.5. Calcium, sulfate, and boron are soluble constituents, which can leach and migrate. The EPA (Rittenhouse 1993) has stated fly ash as non-hazardous.
Design Considerations	Where the groundwater table is high, a drainage blanket should be provided below the fly ash fill to promote a capillary cutoff and prevent frost heave and resiliency of the subgrade. Runoff from paved surfaces should be discharged into a drainage system. Surface waters from peripheral areas should be diverted away from the embankment to minimize infiltration into the fly ash. The side slope of embankments should be covered with at least 2 feet of soil to prevent erosion. If concrete is to be formed directly on fly ash, a polyethylene barrier should be placed on the fly ash to prevent moisture absorption from the fresh concrete and to serve as a moisture barrier. Use fly ash in the concrete to reduce sulfate attack.
Construction Considerations	Fly ash behaves like silt, thus, dusting will occur when dry, and compaction is difficult when wet. Some means for adding water should be available on site to keep the water content near optimum for compaction. Surface protection to minimize erosion may be required. Compaction is obtained with smooth drum vibratory rollers or self-propelled, pneumatic-tired rollers. Use 10-inch lifts and compact the fly ash immediately after spreading. The use of test strips to develop the most efficient compaction procedures is advisable.

Table 3-9. Air-Cooled Blast Furnace Slag Design and Construction Guidelines

Item	Guideline
Design Parameters	**Compacted Moist Density:** 70 to 94 pcf, varies with size and gradation **Gradation:** Can be graded to any specified size from 4 inches down. **Angle of Shearing Resistance:** 35° to 40° **Permeability and Compressibility:** Depends on final specified gradation. Generally similar to gravel and sand.
Environmental Considerations	Slag contains small amounts of sulfur in combined alkaline compounds. The pH of water in contact with slag is generally in the range of 8 to 12, which tends to inhibit corrosion. Some washing of the aggregate may be required to control the pH to 11 or less to meet AASHTO specifications. There are no known environmental concerns.
Design Considerations	The slag behavior is similar to natural angular gravel and sand deposits. The highest internal stability occurs for aggregate that is well graded with a maximum particle size of 16 inches. The amount passing #200 sieve should be limited to 5 to 7 percent. However, the density increases for well-graded materials. If lightweight fill is desirable, uniformly graded materials should be specified. Absorption in slag is usually in the range of 1 to 6 percent by weight. Slag is highly resistant to weathering and abrasion, and can be placed below the water table and next to lakes and rivers.
Construction Considerations	Slag can be placed and compacted in the same manner as natural gravel and sand.

Table 3-10. Boiler Slag Design and Construction Guidelines

Item	Guideline
Design Parameters	**Dry Density, Loose:** 60 to 78 pcf **Dry Density, Compacted:** 82 to 102 pcf **Optimum Moisture:** 8 to 20% (Stroup-Gardiner and Wattenberg-Komas 2013b) **Angle of Shearing Resistance:** 38° to 42° **Coefficient of Permeability:** 0.3 to 0.9 mm/s **Grain Size Range (Percent Passing):** 90 to 99% on #4, 62 to 89% on #8, 16 to 46% on #16, 4 to 23% on #30, 2 to 12% on #50, 1 to 7% on #100, and 0 to 5% on #200 (Stroup-Gardiner and Wattenberg-Komas 2013b) **Atterberg Limits:** Non-plastic **Compressibility:** Comparable to sand, at same relative density
Environmental Considerations	After 4 days of soaking, the pH of the water solution is generally in the range of 6.7 to 7.0. Barium has been detected by toxicity tests, but at levels well below the EPA specified standard. There are no known environmental concerns with the use of this material.
Design Considerations	The aggregate is durable and satisfies acceptable limits for soundness tests. The aggregate works well as an underdrain filter material, provided the gradation requirements are met. Side slopes should be covered with a minimum of 2 feet of cover material since exposed material has low stability. Specify standard proctor compaction, AASHTO T 99, since some degradation occurs during laboratory compaction in accordance with AASHTO T 180.
Construction Considerations	Compact with several passes of a pneumatic roller or a smooth-drum, vibratory roller. Keep water content at or above optimum water content, as determined by AASHTO T 99. 6 to 10 passes are usually sufficient. Material must be kept wet since there could be a loss in stability when material dries.

5.0 CONSTRUCTION SPECIFICATIONS AND QUALITY ASSURANCE

5.1 Specification Development

A discussion on specification items and guide and/or example specifications for lightweight fill materials are contained on http://www.GeoTechTools.org website. Additionally, guidance on quality assurance methods and means for lightweight fills are contained on this website.

For lightweight fill materials, such as EPS-block geofoam and lightweight concrete, that have inherent strength, the compressive strength is dependent on the plant manufacturing process for geofoam, while that of cellular concrete is usually density related. For these lightweight fill materials, the specifications should include:

- the type of lightweight fill material to be used, including the maximum and minimum densities that are tolerable;
- the minimum compressive and/or flexural strength; and
- the lift thickness for the cellular concrete, or the block dimensions for the EPS-block geofoam.

For lightweight fill materials that behave as a granular material, the specifications should include the following.

- The type of lightweight material that is to be used, including the minimum and maximum density as delivered to the job site.
- The gradation of the fill material.
- Other tests, depending on the type of lightweight fill material used. For example, it might be appropriate to include tests such as durability, as measured by the Los Angeles abrasion test, or the percent absorption of water.
- The lift thickness, the placement, and compaction procedure. For lightweight fill materials that resemble normal soil sizes, such as gravel, sand, and silt, the compaction can be specified in terms of a percent of AASHTO maximum density standards. Where recycled fill materials, such as shredded tires or wood fibers are used, the specification should describe the method of compaction. This would include the type of equipment and the number of passes to be used to induce compaction. If there is any doubt as to lift thickness or percent compaction, consider test strips at the beginning of the field operations.
- The method of payment. A suggested procedure would be to base the payment on the amount of fill to be supplied in cubic meters on the basis of the planned final cross

sections and the existing ground surface. Under this method, shrinkage would be the responsibility of the contractor.

The lightweight fill materials that fall into this granular category include wood fiber, blast furnace slag, fly ash and boiler slag, ESCS, and tire shreds.

5.2 Quality Assurance and Construction Control

For the cohesive lightweight fill materials, field monitoring should include measurements of the density and compressive strength of the materials supplied. For EPS-block geofoam, the density and compressive strength will be a function of the grade delivered with appropriate manufacturer quality control documentation. Some manufacturers use a coloring scheme to differentiate between grades of EPS geofoam blocks. Samples should be obtained for quality assurance testing. Observations of the placements of the blocks should also be made to confirm that the blocks are placed without continuous joints and that shear transfer plates are installed between successive lifts of the blocks. The geomembrane, or concrete, covering the blocks should also be measured to confirm thickness and complete enclosure of the blocks. The seams within the geomembrane should be sealed properly, and quality assured. See NCHRP 529 report (Stark et al. 2004a) and the accompanying NCHRP Web Document 65 (Stark et al. 2004b) for detailed discussions and guidance on construction, construction control, and quality assurance with EPS-block geofoam.

For foamed concrete, the ingredients are mixed directly at the job site and then pumped to the location for use. Samples of the freshly mixed fill should be obtained at the point of placement in a manner similar to concrete testing for performance of density and compressive strength. The lift thickness of each pour should be measured to ensure that it does not exceed the maximum specifications and to confirm there is sufficient hardening of one lift before the next lift is placed. The material must support foot traffic prior to casting subsequent lifts.

For the granular lightweight fill materials, monitoring and construction control generally follows the same procedures used for conventional soil placement and compaction. This would include monitoring of the lift thickness, number of passes, and moisture content at the time of placement, and degree of compaction. A check should also be made on the gradation of the material being supplied, as well as any contamination with undesirable materials.

In the case of tire shreds or wood fibers where compaction tests are not appropriate, field monitoring would include measurement of the lift thickness and the number of passes with the compaction equipment. Visual observations or measurements of the resiliency of the deposit during the multiple passes will also aid in determining whether additional passes of the compaction equipment are necessary. Test sections at the start of construction are desirable to either confirm the specified number of passes to make modifications at an early stage in the project. See NYSDOT GEM-20 (2015a), NYSDOT GCP-19 (2015b), and CalRecyle Usage Guide (Cheng 2016) for detailed discussions and guidance on construction, construction control, and quality assurance with tire shred fill.

For projects where the tire shreds will be used, field monitoring should also be undertaken to confirm that material specifications are met, such as size and that there is not excessive steel wire with the tire shreds or that materials other than rubber are being supplied. For projects where wood fibers are used, the gradation of the wood fiber should be checked on a daily basis to confirm that there is a blend of coarse sizes to the sawdust sizes. Only fresh wood fibers should be used to build the fills, so as to prolong the life of the fill. The Washington State Department of Transportation (Kilian and Ferry 1993) has provided a classification for visually identifying different degrees of decomposition in the wood fiber. Project personnel should be provided with this classification system so as to exclude the partially decomposed wood fibers from the new fill.

6.0 COST DATA

6.1 Cost Components

Costs for lightweight fill materials are highly variable and will depend upon a number of factors:

- The basic cost of the material. If the material is a waste product that can be used without additional recycling, such as wood fiber, the cost will be relatively low. Recycled materials are also relatively cheap, but crushing or shredding and sieving will increase the cost slightly.
- Transportation costs. If the project is located relatively far from the supply source, the transportation costs could be significant. Mode of transportation will also affect the price. For example, a lightweight slag may not be produced locally might be cost effective it can be barged, in lieu of long-haul trucking, close to a project site
- Quantity of material. The amount of material to be used on the project, and the staging of use, can affect the average transportation cost and may affect the basic material cost, particularly for smaller quantities.
- Availability of materials. Some materials are not readily available in some regions, e.g., blast furnace slag. If the materials are produced in very low quantities, there may not be sufficient materials available, unless multiple sources are used for the product.
- Placement and/or compaction costs could be higher than for soil fill. This would include moisture control for fly ash and boiler slag, geomembrane or concrete covers for geofoam, or greater numbers of compaction passes for shredded tires.

When calculating the costs for most lightweight fills, a common denominator should be used. Generally, prices for granular materials are quoted by suppliers as a cost per ton of material. However, the density of lightweight materials varies considerably and the loose versus compacted density of some lightweight materials varies. For example, the density of TDA as stockpiled and shipped ranges from 25 to 35 pcf, while the in-place compacted density ranges from 45 to 50 pcf (Cheng 2016). A ton of lightweight fill material will provide a much greater volume than conventional soils with a higher density. For this reason, the cost comparison should be made on the price per in-place cubic yard. The conversion from dollars per ton to dollars per in-place cubic yard can be made with an estimated in-place density and a delivered cost per ton price. Note that the dollars per ton is frequently given as the price FOB at the plant or processing facility and the transportation costs must be added to this number. This variation of cost ranges; at source, delivered, and in-place; is illustrated in Table 3-11 for EPS-block geofoam.

Table 3-11. Typical Cost Ranges for EPS-block Geofoam at Source, Delivered, and In-Place

Material	Material Cost/yd^3 FOB at source	Delivered Material Cost/yd^3 FOB at Project	In-Place Cost/yd^3
EPS-block geofoam	\$40 to \$80	\$40 to \$80	\$40 to \$100

The benefit of one type, or density, of lightweight fill versus other types is another cost and design consideration. For example, use of various types and densities of lightweight fill for stabilization of a landslide can result in different computed stability safety factors, and level of risks. Or, the volume of lightweight fill could be different for the various material densities to achieve similar stability safety factors.

Typical unit prices that are reported in the literature for various types of lightweight fill materials are summarized in Table 3-12.

Table 3-12. Typical Cost Ranges for Lightweight Fills

Lightweight Fill	Material Cost/yd^3 FOB at source	Delivered Material Cost/yd^3 FOB at Project	In-Place Cost/yd^3
EPS-block geofoam	\$40 to \$60	\$60 to \$125	\$40 to \$100
Cellular concrete	n/d	\$70 to \$150	\$250 to \$340
Wood Fiber	n/d	\$6 to \$26	\$8 to \$30
Air-cooled blast furnace slag	\$6 to \$8	n/d	n/d
Expanded blast furnace slag	\$11 to \$15	n/d	n/d
Boiler slag	\$2 to \$3	n/d	n/d
Fly ash	\$12 to \$16	n/d	n/d
Expanded shale, clay, and slate (ESCS)	\$30 to \$45	n/d	n/d
Tire shreds	n/d	\$15 to \$30	n/d

n/d = no data

In some cases, available cost information is limited, and the prices could vary significantly from that shown in the table. These prices were generated from recent literature, suppliers, and user agencies, and will vary due to the factors listed above. The prices vary considerably, due to factors previously discussed. Note that in-place tire shred fills as low at \$2/yd^3 have been reported, and that tire shred costs may be significantly offset with state recycling incentives.

Engineers should contact local suppliers to obtain specific prices, or price ranges, applicable to their region served by their agency. Again, transportation costs should be added to estimate a delivered unit cost of the lightweight fill on a specific project. An example cost comparison of various fills for an embankment fill project in Milpitas, California (Cheng 2016) is presented in Table 3-13.

Table 3-13. Example Lightweight Fill Project Cost Comparison

Material	Total Cost*
Traditional Soil Fill	$563K
Pumice Rock	$633K
EPS-block Geofoam	$1,145K
Expanded Shale Clay	$490K
Wood Chips	$545K
Tire-derived Aggregate	$334K

* Total costs based on material, transportation and longevity costs; does not include installation costs or contractor's overhead and profit.
Source: Cheng 2016

Additional cost considerations would include compaction methods, loose versus compacted volume adjustment, and any – a specialty items, such as the need for geotextiles, geomembranes, drainage blankets, or soil cover.

7.0 CASE HISTORIES

Four case histories are presented to illustrate the various types of applications for which lightweight fill materials can be used in highway construction. Each project description provides information on the setting, the design considerations, the construction procedures, and the performance data.

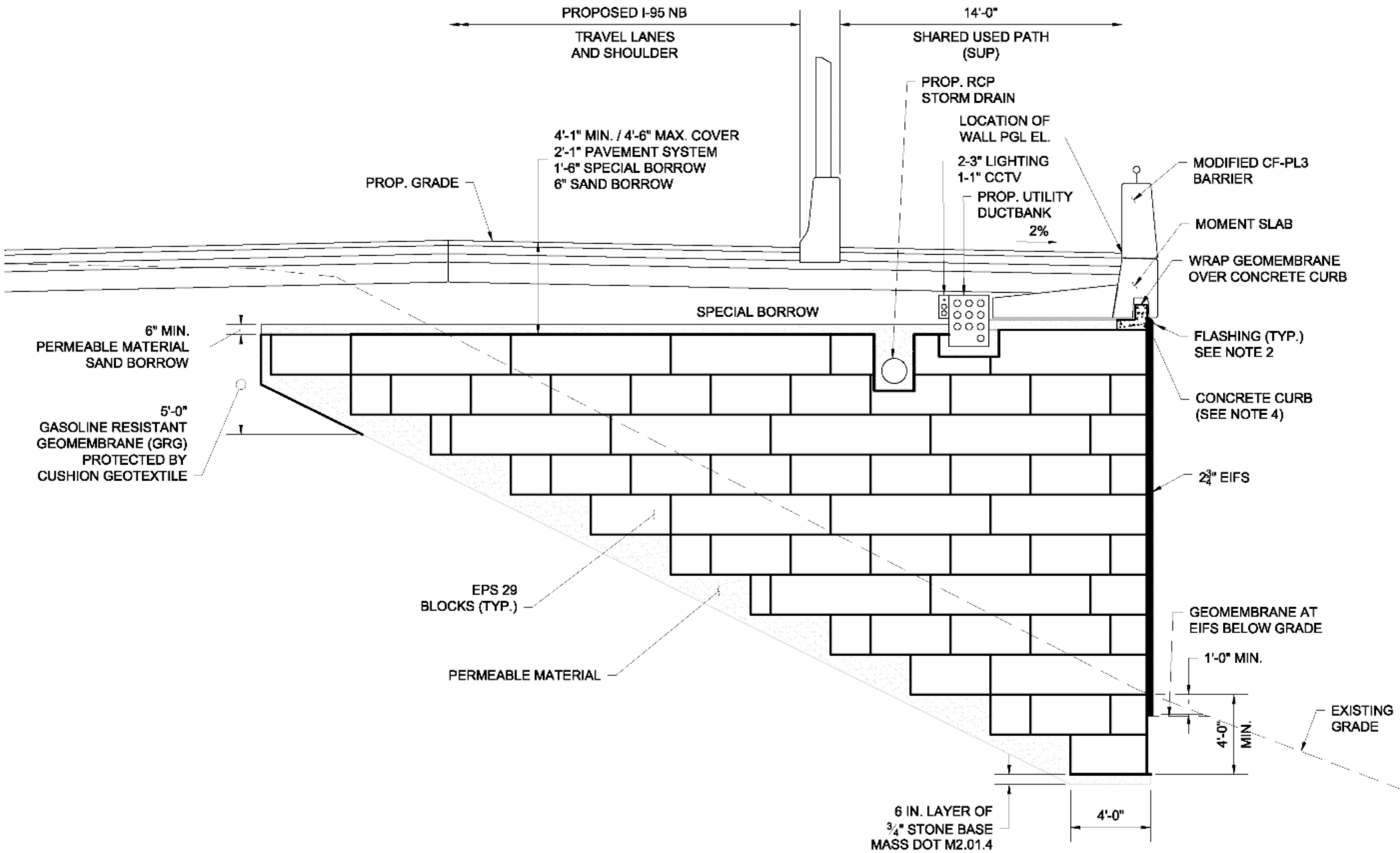
These case histories provide a brief synopsis, and the reader is referred to the technical literature for detailed discussions of these projects. References are listed for each case history.

7.1 GeoFoam

The Massachusetts Department of Transportation (MassDOT) used EPS-block geofoam on their Whittier Bridge/ I-95 Improvement project. The existing through-truss Whittier Bridge constructed in 1951 is being replaced with a network tied arch structure. Each of the two parallel structures will carry four travel lanes, a high speed shoulder, and a breakdown lane. A shared-use path will be constructed on the northbound side to facilitate connections to walking and cycling trails. This new bridge is a signature project of the MassDOT Accelerated Bridge Program.

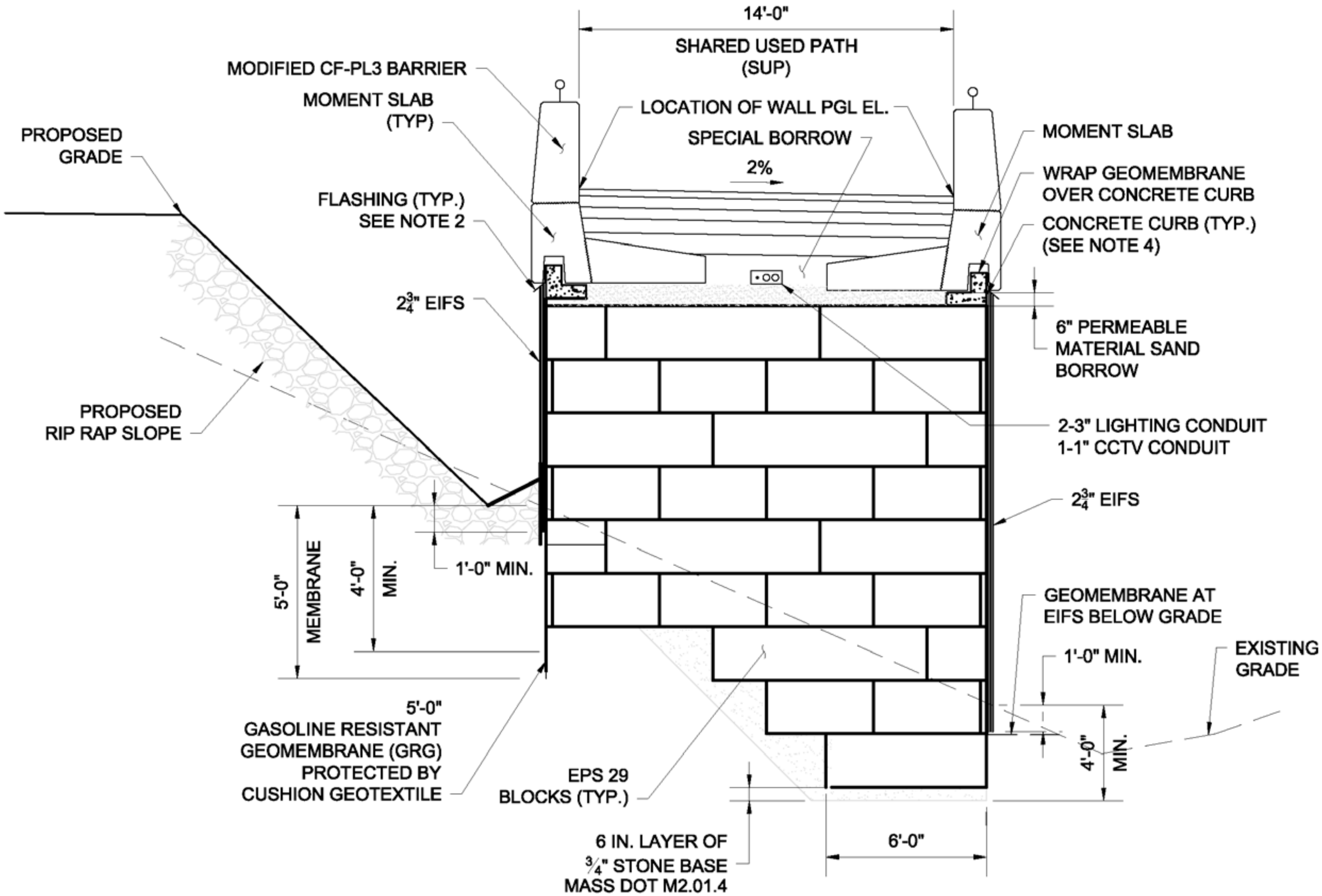
MassDOT employed several innovative construction methods to expedite project schedule, ensure high quality results and long lasting infrastructure, and reduce maintenance costs. One such technique for the Whittier Bridge/I-95 project is the use of EPS-block geofoam lightweight fill material. The blocks are being used for the base of the northbound breakdown lane, the right northbound travel lane, and the shared-use path. These areas are underlain with soft organic and clay soils. If the EPS blocks were not used, the contractor would have to excavate down 30 feet to replace the unsuitable soils, and install a drainage system as well, before widening the highway. The use of EPS results in significant construction advantages in both time and cost savings.

Typical EPS block fill cross section on the embankment widening and on the ramp are illustrated in Figures 3-10 and 3-11, respectively.



Massachusetts Department of Transportation

Figure 3-10. Whittier Bridge typical embankment widening section.



Massachusetts Department of Transportation

Figure 3-11. Whittier Bridge typical ramp cross section.

EPS block placement on the embankment widening, and going around utilities, is shown in Figure 3-12.

Massachusetts Department of Transportation

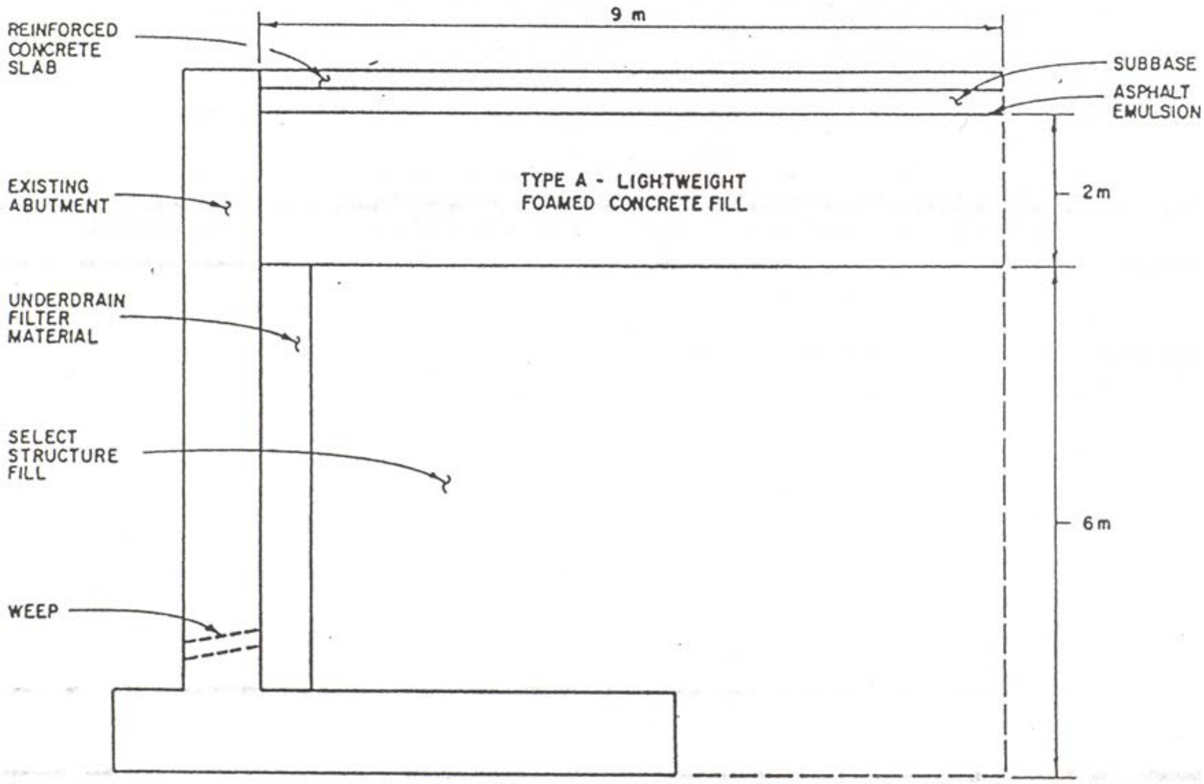
Figure 3-12. Whittier Bridge EPS block placement on embankment widening.

7.2 Foamed Concrete

The New York State Department of Transportation (NYSDOT) used foamed concrete fill on 12 separate projects. The typical application was for balanced excavations (Harbuck 1993). In these procedures, the existing soils were excavated to a depth required to balance the weight of the lightweight foamed concrete fill, which would be placed to a grade higher than existing. Thus, no additional load was applied to the foundation soils. Generally, 3 feet of foamed concrete fill at a density of 30 pcf will have the same weight as 9-inch of existing fill with a density of 120 pcf.

A typical example of weight balancing was for a two-span structure carrying Route 150 over Amtrak in Rensselaer County that was replaced with a single-span structure. The abutments were structurally sound and were used to support the new superstructure. It was necessary to raise the grade of the approach embankments by 3 feet. This is not a large grade increase, but the embankments were underlain by 30 to 33 feet of very soft-to-soft clay that in turn was underlain by loose silt, eventually grading to a very compact silt. An analysis of the bearing capacity and slope stability of these soils indicated that the raising of the grade could jeopardize the factor of safety.

To reduce the loading on the soft soils, the upper 6 feet below final grade was constructed of foamed concrete with a maximum unit weight of 30 pcf. A drain was placed behind the existing abutment so as to reduce water pressures on the abutment. The final design is shown in Figure 3-13.



Harbuck 1993, TRB

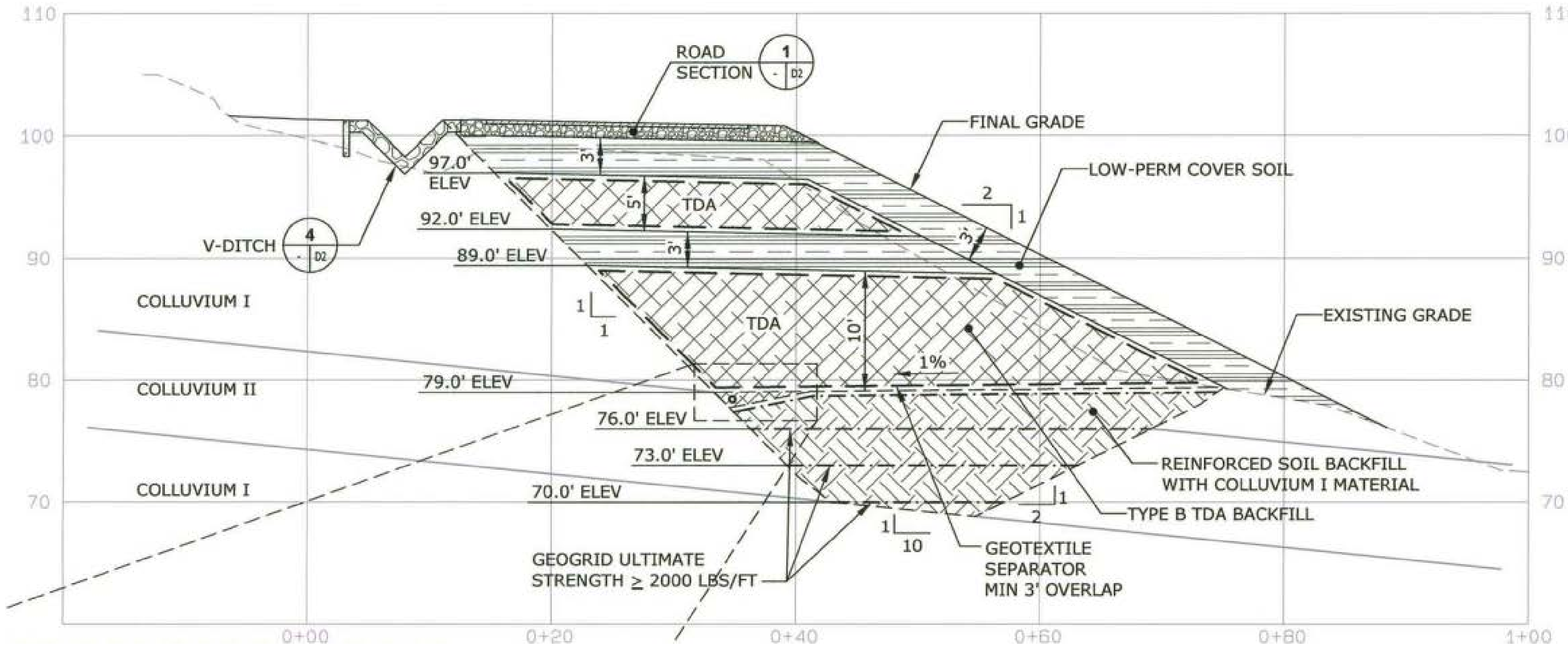
Figure 3-13. Foamed concrete fill behind existing abutment, New York project.

7.3 Tire Shreds

The CalRecycle Usage Guide, Tire-Derived Aggregate (Cheng 2016) provides several case histories of TDA use in California. Applications include embankment construction over soft soils, roadway landslide stabilization, retaining wall backfill, light rail vibration damping, landfill drainage, and leach fields. The TDA application in the following project summary (http://www.calrecycle.ca.gov; Cheng 2016; Kennec, Inc. 2007) is the use as lightweight fill to stabilize an existing embankment.

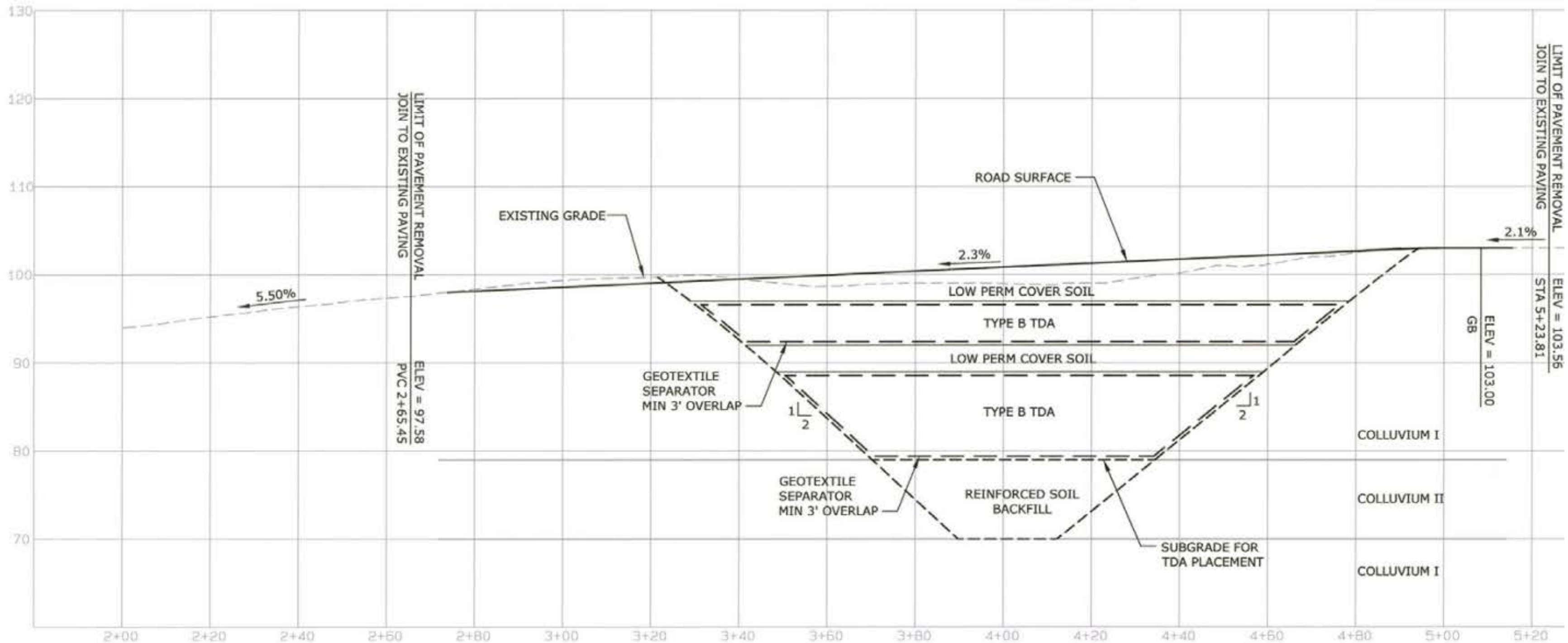
A 160-foot long section of Marina Drive in Calpella, California had been gradually settling and moving since the 1960s. The settlements were repaired over the years by filling with base course material and asphalt. The repeated repairs led to a total thickness of base course and asphalt of about 7 feet. This accumulating weight increased the destabilizing loads of the slope.

A design to stabilize the slope with TDA lightweight fill replacement and soil reinforcement was engineered (Kennec, Inc. 2007). A slope cross section illustrating this repair plan is shown in Figure 3-14. The longitudinal, along the roadway section is shown in Figure 3-15.



Kennec, Inc. 2007

Figure 3-14. Marina Drive slope cross section.



Kennec, Inc. 2007

Figure 3-15. Marina Drive longitudinal section of fill replacement.

The fill replacement extended to the underlying Colluvium I. Fill consisted of compacted, reinforced backfill; low permeability cover soils; and two zones of TDA fill. The lower zone of TDA was 10-foot thick, which is the maximum recommended fill height (see Table 3-5). The upper and lower TDA fill zones were separated by a 3-foot thick layer of low permeability soil. The TDA fills were wrapped in a geotextile, that functions as a permeable separator between the finer graded soils and more open graded TDA fill.

A construction photograph is presented in Figure 3-16, and a view of the completed repair is presented in Figure 3-17.

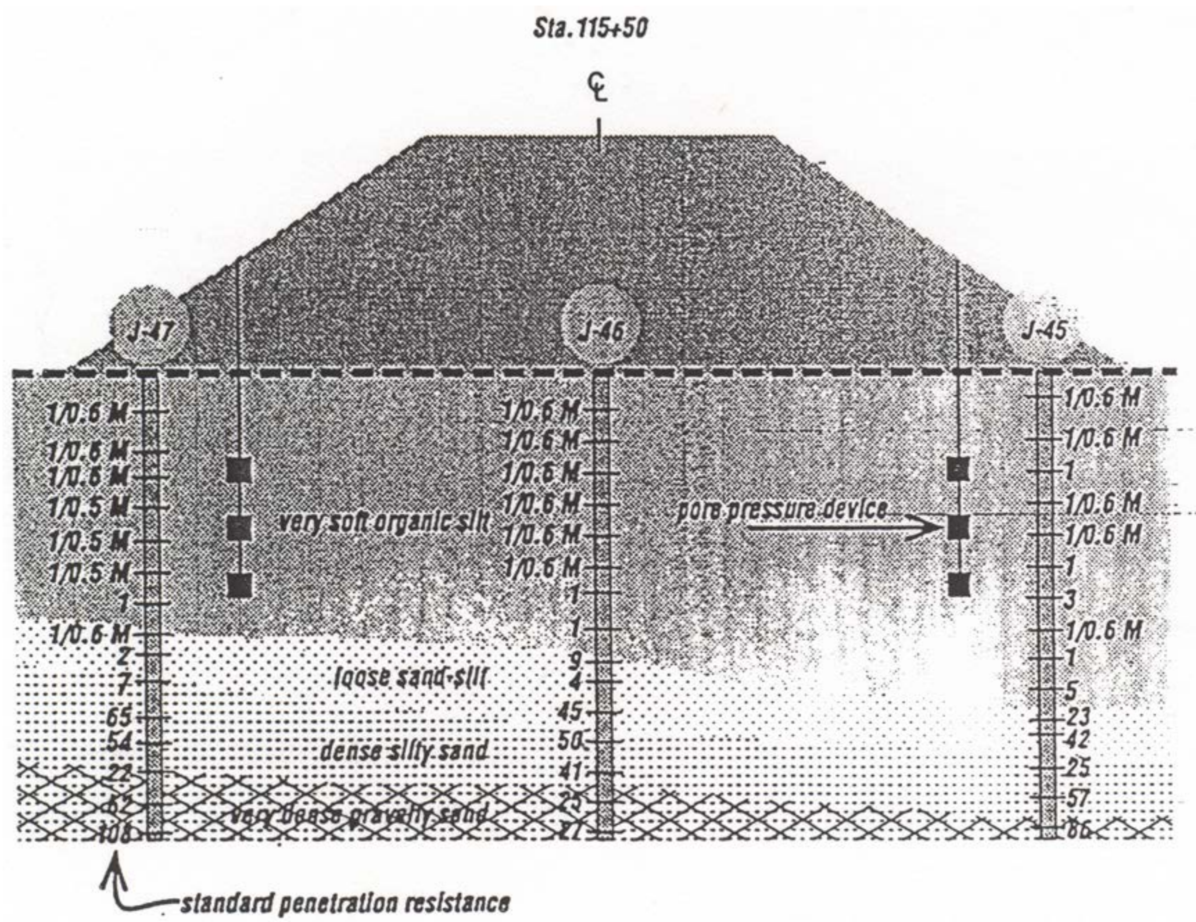
Figure 3-16. Placement and compaction of TDA within geotextile wrap.

Figure 3-17. Completed Marina Drive repair.

Design included a surface water management plan and controls. Approximately 133,000 waste tires were used in the two lightweight fill embankment zones. This constructed repair option provided an overall cost savings of $740,000 over the next option.

7.4 Wood Fiber Embankment

Approximately 600 feet of a two-lane roadway was constructed over swampy terrain in Washington State in 1987-88 (Allen and Kilian 1993). A generalized soil profile at the site, including the existing and final roadway grades is shown in Figure 3-18.



Allen and Kilian 1993, TRB

Figure 3-18. Soil cross-section at wood fiber fill location, Washington project.

The standard penetration resistance values in the organic sandy silt were generally less than 1. In place water contents ranged from 94 to 364 percent, with an average of 172 percent. The liquid limit ranged from 61 to 90 percent, and the plastic limit from 53 to 65 percent. The average unconsolidated, undrained strength determined from triaxial shear testing was 150 psf.

The new embankment vertical grades required up to 44 feet of fill. Side slopes were limited to 2H:1V to limit the right of way and the fill volume required and to minimize the amount of wetlands removed by construction of the embankment. Slope stability calculations indicated that construction of the embankment using granular soil was not practical. By assuming no strength gain in the foundation soil, the maximum height of granular fill that could be safely constructed was determined to be 12 feet. If the embankment were constructed slowly, allowing pore pressure dissipation and strength gain, it was estimated that 20 months would be required for construction of the embankment. In addition, settlement under 44 feet of granular fill was expected to be 8 feet, which was unacceptable because of the necessity of installing a culvert at the base of the fill.

In order to obtain a minimum factor of safety of 1.25 for slope stability and 1.5 for bearing capacity, it was necessary to use a lightweight fill material for the embankment. Wood fiber was available in this area and was selected for the embankment fill. The initial 5 feet of fill consisted of a silty, gravelly sand to raise the base of the embankment above the prevailing groundwater table. The upper 4 feet of the fill was also a granular deposit to provide adequate subgrade support for the roadway. The remaining 35 feet of the embankment height consisted of the wood fiber-fill. For the stability calculations, the density of the wood fiber-fill was used as 38 pcf and an angle of 40°. The granular soils were assumed to have a unit weight of 125 pcf and an angle of internal friction of 37°. Five layers of geotextile reinforcement were embedded within the lower granular fill, as well as within the wood fiber, so as to meet the minimum factor of safety requirements for slope stability and lateral spreading considerations. The geotextile reinforcement was needed only until the soil gained sufficient strength to support the fill without reinforcement. It was estimated that this would require about 8 months, and the geotextiles were designed allowing a relatively high creep limit of 60 percent of ultimate tensile strength.

Embankment settlement using the wood fiber fill was estimated to be 5 feet, plus an additional 0.4 feet of secondary consolidation over a 20-year period. An accurate settlement estimate was necessary to determine the amount of additional wood fiber fill necessary to accommodate the anticipated settlement. Re-leveling with granular soils would reduce the slope stability factor of safety.

The as compacted density of the wood fiber fill was determined to be 38 pcf. Compaction was obtained by routing hauling equipment over the entire lift thickness of 1-foot. The minimum mass of the hauling equipment was 33,000 lbs.

Piezometers were installed in the organic silts and monitored as the embankment was placed in height. The maximum allowable pore pressure ratio was equaled or exceeded twice during construction. This occurred when the fill height reached 22 feet and 31 feet. In the first case, fill construction was stopped for 52 days; in the second case, for 130 days.

The total time required to construct the embankment was just under 11 months. The subgrade was reached in September 1987, and paving began in October 1988. The measured settlement of the embankment immediately following construction to subgrade level was 3.3 feet. Just prior to paving, this settlement had increased to 4 feet. In September 1992, settlement increased to 4.6 feet. This compares favorably with the estimated primary settlement of 5 feet.

The cost of the embankment construction, including the geotextile reinforcement, was $972,000. If ground improvement had been undertaken with stone columns, the estimated cost was $1,500,000. The cost for constructing a bridge over the weak ground area was estimated to be $1,700,000.

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Chapter 4

DEEP COMPACTION

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1.0 DESCRIPTION AND HISTORY

1.1 Description

Deep compaction is a category description of technologies that rely on dynamic methods to impart high levels of energy to the ground resulting in improvement of soil properties. Kirsch and Kirsch (2010) divide such methods into compaction by vibration using depth vibrators or vibratory hammers and compaction by impact using drop weights or explosion. These methods are most applicable to loose cohesionless soils having little to no fines content. The dynamic forces imparted to the soils densify the cohesionless soils resulting in increased bearing capacity, increased shear strength, reduced settlement, and increased liquefaction resistance. In this chapter two technologies (methods) are discussed in detail: Deep Dynamic Compaction (DDC) and Vibro-compaction (VC). Deep dynamic compaction densifies materials by drop weights imparting high levels of impact energy at the surface. Vibro-compaction uses specialized vibrators lowered into the loose soils which send out horizontal vibrations to densify nearby materials.

Closely related technologies include blast densification, an alternate method that densifies cohesionless soils through high levels of impact dynamic energy; rapid impact compaction (RIC) that uses an excavator-mounted drop weight to compact soils to depths of 10 to 20 feet; and a group of technologies similar to vibro-compaction termed vibro-replacement, whereby additional material is added to the ground as the dynamic vibrations are imparted. Vibro-replacement methods include vibro-stone columns, vibro-concrete columns and sand compaction piles. These are further detailed in Chapter 5 Aggregate Columns.

Deep dynamic compaction is a method of ground modification that results from the application of high levels of energy at the ground surface. The energy is applied by repeatedly raising and dropping a tamper with a mass ranging from (10 to 40 kips) at heights ranging from 30 to 100 feet. The tamper is lifted and dropped by a conventional crane with a single cable plus a winch that has a free spool attachment that allows the single cable to unwind with minimum friction. The tamper's energy of impact at the ground surface results in densification of the deposit to depths that are proportional to the energy applied. The depth of improvement generally ranges from 10 to 35 feet for light- to heavy-energy applications, respectively. Following the high energy level application, the surface of the deposit is in a loose condition to a depth equal to the depth of the craters. This surface is then compacted on a tight grid basis, with a low-level energy application called an ironing pass.

The deep dynamic process is illustrated schematically in Figure 4-1. The arrows represent energy that is transmitted into the soil mass following impact of the tamper. The predicted depth of improvement is shown as a function of the energy of a single drop.

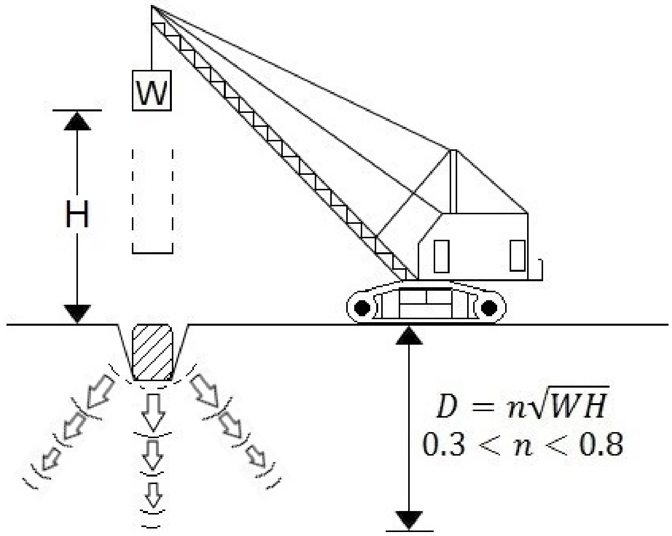


Figure 4-1. Schematic illustration of deep dynamic compaction.

If ground improvement is needed to provide a suitable bearing stratum for an embankment or a structure, deep dynamic compaction may be a viable solution. Deep dynamic compaction is generally the most economical form of site improvement. Conventional cranes are used to lift and drop the tampers, which allow local contractors to compete with specialty contractors to perform this work. Deep dynamic compaction can be used on a wide variety of deposits, ranging from silty soils to boulder-sized granular deposits. This procedure can improve both fill deposits and natural soil deposits.

Vibro-compaction is a ground modification technique that uses specially designed probe-type, depth vibrators for in situ densification of loose sands and gravels, as shown in Figure 4-2.

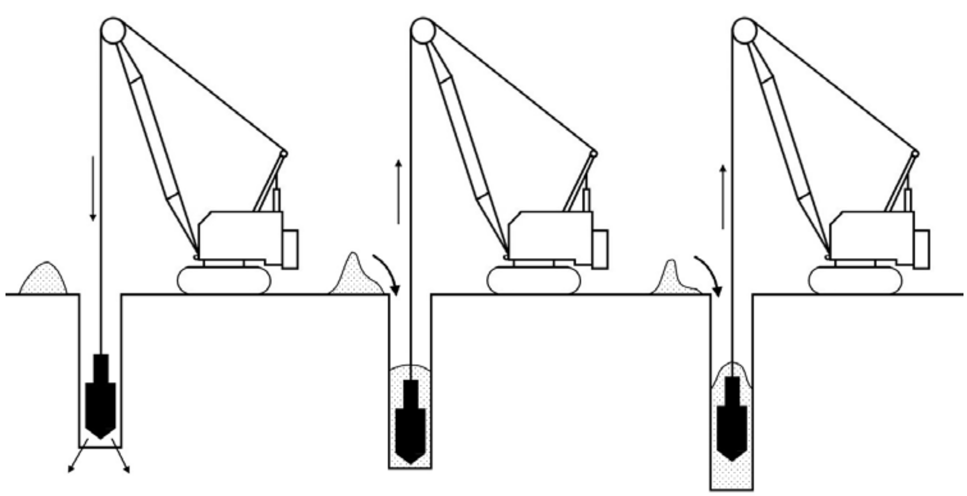


Figure 4-2. The vibro-compaction process.

Originally called vibroflotation, vibro-compaction was accomplished with water jetting, hence the name (Vibroflotation Foundation Co. 1980). Subsequent equipment development featuring higher horsepower and amperage made a dry operation possible. The majority of vibro-compaction projects, however, are accomplished by the jetting water (wet) method, and this chapter reflects this tendency.

The mechanism of densifying granular, cohesionless soils with vibrators can be briefly described as follows: Mechanical vibrations and simultaneous application of water nullify the effective stresses between the soil grains, which are rearranged, unconstrained, and unstressed under the action of gravity to the densest possible state, thus providing permanent compaction. In the immediate vicinity of the vibrator, the soil is saturated, liquefies locally and temporarily under the influence of the vibrations (Moseley and Priebe 1993).

There are numerous natural and man-made deposits where vibro-compaction, can be applied, including densification of granular hydraulic fills, coastal plain sediments, glacial deposits, alluvial soils, and miscellaneous granular fills and/or deposits to permit construction of shallow foundations. Also, liquefaction potential can be reduced by vibro-compacting loose, granular soil to a density beyond the threshold density triggering liquefaction. In earth retaining problems, the process can be performed prior to wall construction to decrease active earth pressure and increase passive resistance as the density is improved. Generally, vibro-compaction can be used to achieve the following results:

- Increased soil bearing resistance, permitting shallow foundation construction
- Reduced foundation settlements
- Increased resistance to liquefaction
- Increased shear strength
- Reduced permeability
- Filling of voids in treated areas

As with any ground modification technique, vibro-compaction has its limitations. The improved in situ soil characteristics depend on the in situ soil type and its gradation, the spacing of the compaction points, the characteristics of the equipment utilized, and the compaction duration. All of these factors affect the outcome of the project.

1.2 Historical Overview

1.2.1 Deep Dynamic Compaction

Impacting soil deposits with tampers dropped from varying heights has been used for centuries. The Romans reportedly used this process to densify loose soils (Kerisel 1985). Cohesionless soils in Germany were compacted with a 4 kip tamper and a 5-foot drop from a steam shovel in 1933 (Loos 1963). The Corps of Engineers experimented with heavy tamping at the Franklin Falls Dam construction site in 1936 (USACE 1938). In 1955, dynamic compaction was used in South Africa to densify loose soils to support a 250 feet diameter crude oil tank (Hobbs 1976). In Russia, heavy tampers were used to compact loessial silty and sandy soils, beginning about 1960 (Bobylev 1963).

Use of deep dynamic compaction began on a regular and continuing basis in Europe in 1969 and in the United States in 1971. In Europe, tampers of 18 to 22 kips were dropped from heights of 25 to 40 feet to densify fill deposits. This process was called heavy tamping, and was generally used in good quality fill deposits, such as rock waste, rubble, and sand. After a few years, the process was expanded to include fine grain soil deposits, and the name was changed to dynamic consolidation (Menard and Broise 1975).

In the United States, densification was initially achieved using tampers in the range of 4 to 12 kips with drops of 20 to 35 feet to densify loose rubble fill and granular deposits to support lightly loaded structures. Later, tampers up to 33 kips were used to densify former landfills. This technique was initially called “pounding,” but eventually became known as dynamic compaction (FHWA 1986). Herein, the term deep dynamic compaction is used to differentiate this method from shallow dynamic compaction methods such as high energy impact rollers.

Before 1975, European and American practices developed independently and were somewhat experimental. In 1975, a technical article by Menard and Broise (1975) was published that dealt with the theoretical and practical aspects of deep dynamic compaction. A formula was presented for predicting the depth of improvement as a function of the applied energy. This article presented sufficient information to place the deep dynamic compaction process on a sound technical basis. Subsequent articles published in Europe and the United States used this paper as the starting point to expand and exchange the knowledge base of deep dynamic compaction. In the United States, the state of practice was summarized by Lukas in ”Dynamic Compaction for Highway Construction” (FHWA 1986) which presented guidelines for implementation. Guidelines for implementation have been kept current in FHWA publications (FHWA 1986, GEC 1 1995) and this manual.

1.2.2 Vibro-compaction

The fundamental concept of vibro-compaction ground treatment was developed in Germany during the early 1930s for compaction of variable and loose naturally occurring sand deposits to depths of 65 feet. It was used in the densification of underwater sands for the seaport developments of northern Germany. Previous methods of compacting sand deposits consisted essentially of surface vibration or rolling. To overcome the limitations of these methods, a technique was developed whereby a metal tube or probe, which had within its lower end an electric motor driving an eccentric weight, was inserted into the ground. Vibrations were imparted into the ground as the tube was inserted to a required depth. To assist penetration of the vibrator, pressurized water was jetted through the tip of the probe. This original process was patented under the name Vibroflotation.

In 1948, the first U.S. vibro-compaction project was performed in Cape May, NJ. By the early 1970s, vibro-compaction was generally accepted as the premier method of densifying deep deposits of sand. Since that time, vibro-compaction techniques have been improved and utilized to solve various types of geotechnical problems involving loose, granular soils, ranging from foundation settlement to poor bearing capacity.

There are several systems that have been historically identified as vibro-compaction. These include vibroflotation, Terra-Probe®, and the vibratory beam. Vibroflotation refers to compaction by means of a vibroprobe, sometimes called a vibroflot that densifies with predominantly horizontal vibrations, while other techniques such as Terra-Probe®, Vibro-Wing®, and Tri-Star® or Y-Probe®, utilize a top pile vibrator that densifies with predominantly vertical vibrations, which normally require closer spacing and are less efficient (Welsh 1986 and Schaefer 1997).

The Great Alaska and Niigata earthquakes of 1964 brought the issue of liquefaction of soils to the forefront. As a consequence, procedures to estimate the seismic response of level ground, embankments, and slopes have been improved. More recently, the extensive geotechnical failures during the Loma Prieta, Northridge, and Kobe earthquakes have served to highlight the importance of ground improvement for seismically unstable sites. At Kobe, observations of sites where a vibro-densification process had been used indicated that while areas outside of the improved sites showed significant evidence of liquefaction in the form of settlements and sand boils, the improved ground either precluded liquefaction or limited deformations to a minimum.

1.3 Focus and Scope

Deep dynamic compaction and vibro-compaction are mature, well-established methods of ground modification. The purpose of this chapter is to provide an overview of these methods and to present the state of the practice in their utilizing for improvement of site soils. This chapter addresses feasibility considerations, including applications, advantages, limitations, and potential disadvantages; construction equipment and required materials; preliminary design concepts; specification and quality assurance development; cost information; and case histories.

1.4 Alternative Technologies

A number of alternative methods of compacting ground in situ exist. Many of these techniques also densify cohesionless soils with some adding structural support to the ground. These techniques are as follows:

1.4.1 Frankipaction

The Frankipaction system is essentially an adaptation of the Franki pile to achieve compaction. The Franki pile is a driven, enlarged-base pile. The pile casing is positioned on the ground and a charge of dry concrete is placed within the bottom of the casing. A drop hammer is used to drive the concrete, which then forms a dense plug that penetrates the ground, dragging the casing down with it. When the pile has reached the desired depth, the concrete end-plug within the casing is driven down and outward to form the pile base. Successive charges of concrete are then placed and compacted as the casing is withdrawn, forming the pile shaft. In Frankipaction, sand or gravel is used instead of concrete, and a compaction pile is created. The major advantage of this system is the ability to put large volumes of stone in layers of soft or compressible soils to minimize settlements or potential shear failures of these layers, while still achieving densification in the granular layers.

1.4.2 Blast Densification

Blast densification has been used since the mid-1950s. While it is potentially economical compared with other systems, it is generally only useful where densification is necessary over large areas and at great depths (>100 feet). The technique has an inherent potential to have a negative effect on both the natural and man-made environment. Blast densification is more vulnerable to litigation than most other densification techniques and therefore requires a high degree of expertise and experience in its execution. Despite these drawbacks, the technique should be considered useful under the right circumstances. Washington State DOT has reported a blast densification project in which potentially liquefiable, loose debris avalanche deposits from the 1980 eruption of Mt. St. Helens was densified by blasting techniques (Kimmerling 1994).

1.4.3 Rapid Impact Compaction

Rapid impact compaction uses a 5 to 9 ton excavator-mounted, weight that is dropped about 4 feet on to a 5-foot diameter tamper capable of imparting 40 to 60 blows per minute. The resulting force of this RIC process densifies soils to depths of up to 10 to 20 feet. The depth of compaction is dependent on the compaction energy level, soil properties, and groundwater conditions.

1.4.4 Compaction Grouting

Compaction grouting is a method in which cohesionless or weak soil, soil with fractures and air pockets, or soil that has settled, is densified using a thick, low-slump grout. The grout forms a bulb at the tip of the grout pipe, displacing the soil. Soil between the grout bulbs is thus compacted and strengthened. The common applications of this technique are described in the Chapter 8, Grouting. The use of compaction grouting as a densification tool is relatively new. Although vibro-compaction and deep dynamic compaction are significantly more economical in granular soils, compaction grouting might prove economical in finer grained soils or layered soils where strength gain is necessary. Compaction grouting also has economic advantages where only localized layers at depth need treatment.

1.4.5 Vibratory Hammer Probe

The vibratory hammer probe method differs from the vibro-compaction method in that the vibrations are transmitted vertically down the attached pipe of a typical diameter of 2.6 feet. The vibro-compaction method transmits a horizontal vibration over a distance of up to 13 feet. The frequency of vibration of the vibratory hammer probe and the location of the vibrator result in a less-effective densification process and therefore must be used on significantly closer pattern spacing. Usually this results in higher overall cost for densification.

1.5 Alternative Solutions

Prior to deciding upon ground improvement by densification, alternate foundation solutions should be investigated to compare construction time and costs. Aside from relocating the project to a more favorable location, common foundation alternatives include the following:

- *Foundations of piles or drilled shafts to transfer the loads to deeper levels.* A structural slab or a geosynthetic-reinforced thick, crushed-stone mat may be required at the ground surface to transfer embankment loading into the deep foundations (see Chapter 6).

- *Excavation and replacement of the weak ground.* Sometimes this solution may also entail use of sheeting and bracing in confined working areas to retain the adjacent soil mass or dewatering when the water table is high. Contaminated soil deposits could result in a very high excavation cost if the spoil must be taken to a special waste site.
- *Preloading of the weak deposits to increase strength and decrease compressibility.* This solution frequently can result in time delays of 6 months to one year to allow the preloading to be effective. Wick drains could be used to accelerate the consolidation process. If materials for surcharging are not available on site, they must be imported and then excavated and removed after the preloading is completed (see Chapter 2).
- *Vibro-replacement could be considered* to stiffen clayey soil deposits (see Chapter 5).
- *Deep soil stabilization techniques,* such as jet grouting, deep soil mixing, or compaction grouting, can be considered (see Chapters 7 and 8).

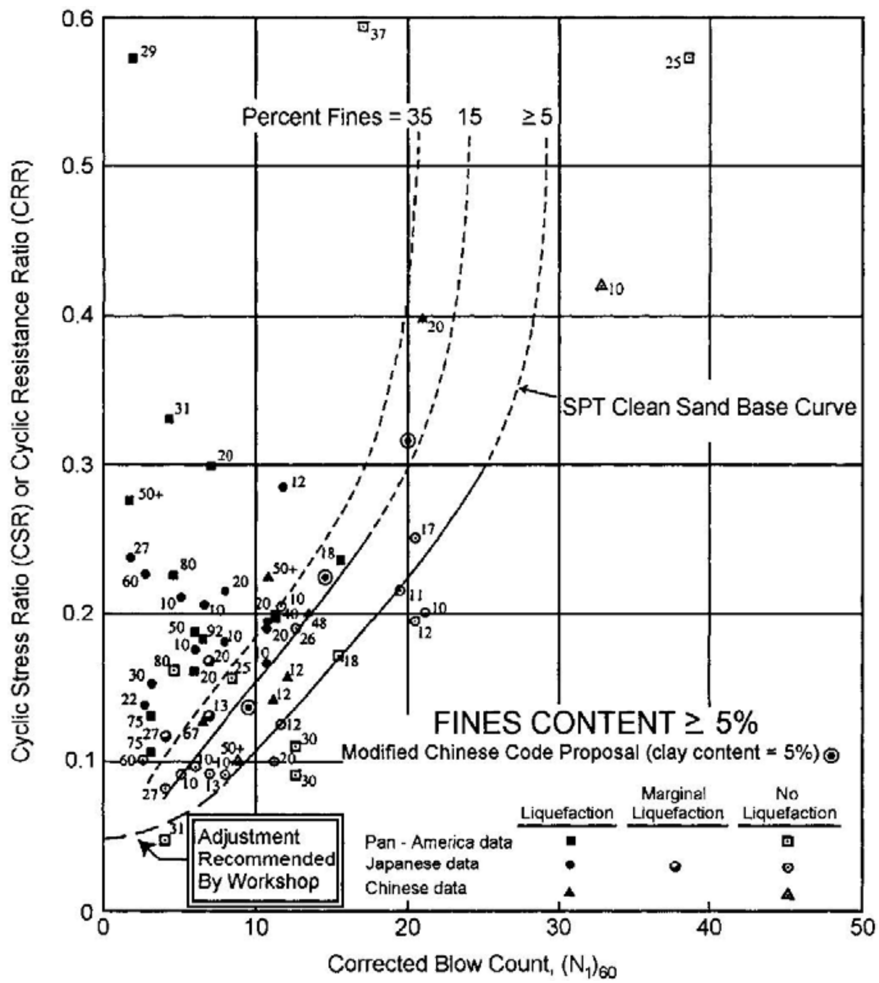
1.6 Liquefaction Potential and Mitigation Assessment

The identification of liquefiable soils is beyond the scope of this manual, but typically they are loose sand deposits either natural or man-made. The chief strategy to improve loose sand deposits to resist liquefaction is to increase their density and/or contain the liquefiable deposit. In developing design guidelines for highway structures and embankments, Cooke and Mitchell (1999) provide the following recommendations:

- Improvement of liquefiable soils should extend to the bottom of the liquefiable material and extend laterally to a distance equal to the depth of treatment.
- Improvement for reducing lateral deformations of embankments is more effective when the foundation is treated in a zone between the crest and the toe of the embankment.
- Field performance suggests that the effect on structures will be minor when the supporting ground is improved to the “no liquefaction” side of liquefaction potential curves, recommended in the simplified method for liquefaction potential assessment.

The deep compaction techniques of DDC and VC may be used at sites with soils that may be susceptible to liquefaction during earthquakes. Saturated sands, silty sands, sandy silts, and silts are likely to be in this category. When DDC or VC are used to densify soils for the support of embankments and structures, it is also necessary to confirm that there will not be a risk of liquefaction or other ground disturbance that could lead to loss of support and lateral spreading. The initial assessment of whether the soil at a site will liquefy in an earthquake is made in terms of whether the in situ shear strength under cyclic loading, represented as a Cyclic Resistance Ratio (CRR), is less than the cyclic shear stress that will cause liquefaction, termed the Cyclic Stress Ratio (CSR).

Combinations of CSR and strength of the soil layer, usually determined in situ by means of penetration tests, have been found that define the boundary between liquefaction and no liquefaction over a range of peak ground motion accelerations. This boundary has been determined through extensive analyses of case history data from many earthquakes. Standard Penetration Tests (SPT), Cone Penetration Tests (CPT), and Becker Penetration Tests for soils containing gravel and cobbles (BPT) are used to determine the CRR. Values of CRR are defined by the points on the boundary curve that separates liquefaction and no liquefaction zones on a plot of CSR vs. penetration resistance or shear wave velocity corresponding to the measured and corrected in situ property. An example of such a plot for liquefaction analysis using the SPT is shown in Figure 4-3.



Youd et al. 2001

Figure 4-3. SPT liquefaction chart for magnitude 7.5 earthquakes.

Thus, if a site underlain by saturated clean sand has a corrected blow count $(N_1)_{60}$ of 10 blows per foot and the anticipated cyclic stress ratio under the design earthquake is 0.25, the soil will liquefy unless the normalized penetration resistance $(N_1)_{60}$ is increased to greater than 22 blows per foot by densification, or the cyclic stress ratio is reduced by transferring some or all of the dynamic shear stress to reinforcing elements. Similar plots are available in terms of normalized CPT tip resistance q_{c1N}. In each case the penetration resistance is normalized to an effective overburden stress of 1 atmosphere.

Although straightforward in concept, the liquefaction potential analysis is complex in application, because (1) the CSR depends on the input motions within the soil layer which, in turn, depend on such factors as earthquake magnitude and intensity, distance from the epicenter, geologic setting, rock conditions, and soil profile characteristics, (2) the CRR depends on such factors as overburden stress, fines content of the soil, and static shear stress, and (3) determination of normalized values of the penetration resistance involves several corrections to the measured values, especially in the case of the SPT.

Information about input ground motions can be obtained from local experience and recorded ground motions near the site, if available, or from seismicity information obtainable from the following sources:

- United States Geological Survey Ground Motion Calculator (https://earthquake.usgs.gov/hazards/designmaps/), which can be used to obtain peak rock accelerations for the site
- USGS Earthquakes Hazards Program website (https://earthquake.usgs.gov/hazards/), which provides design ground motions for buildings and bridges; interactive fault maps; scenarios of ground motions and effects of specific hypothetical large earthquakes; and seismic hazard maps and site-specific data which includes a Beta version of an unified hazard tool that enables determination of site-specific ground motion parameters.

Widely used liquefaction correlation diagrams for SPT and CPT, along with discussions of how to make the necessary computations to obtain the CSR, $(N_1)_{60}$, q_{c1N}, and the CRR are given in Youd, et al. (2001) and Idriss and Boulanger (2008).

The usual design procedure for ground improvement to mitigate liquefaction using DDC and VC is to require that the soil be densified sufficiently to attain a factor of safety against liquefaction triggering, defined by CRR/CSR, greater than 1.5, with a minimum of 1.3, although no single value may be suitable for all conditions owing to the many factors that influence each specific site and problem. Each case needs to be judged on its own merit in the event there are a few points where the safety factor criteria are not met. A few scattered locations where the safety factor is below the minimum is quite different from several low values that are grouped closely together. If a value fails by a large amount it is more significant than if it fails to meet the minimum by a small amount. Settlement attributed to earthquake shaking can be estimated using methods outlined in FHWA GEC 3 (1997).

1.7 Glossary

1.7.1 Deep Dynamic Compaction

This glossary describes terminology unique to dynamic compaction.

Applied energy – Average energy applied at ground surface, which is calculated on the basis of the sum of all the energy applied by dynamic compaction divided, by the surface area of the densified soil.

Contact pressure – The weight divided by the base area of the tamper.

Crater – Depression in the ground at the drop point location that results from energy application.

Depth of improvement – Maximum depth to which measurable improvement is attained.

Drop energy – Energy per blow, which is calculated on the basis of the tamper mass, multiplied by the drop height.

High-level energy – Energy applied to cause densification to the depth of improvement.

Induced settlement – Average ground settlement following densification, which is determined by elevation readings taken before and after dynamic compaction.

Low-level energy – Energy applied to compact the surface deposits to the depth of crater penetration following high-level energy application. Low-level energy application frequently is called the ironing pass.

Pass – The application of a portion of the planned energy at a single drop point location. Multiple drops are required to deliver the energy at each drop point. If all the drops cannot be applied at one time because of deep craters or excess pore water pressures, another pass or passes will be required after excess pore water pressures dissipate, or the craters are filled with granular fill. There is generally a waiting period of at least a few days between passes.

Phase – Describes the pattern in which the energy will be applied. For example, every other drop point of the grid pattern could be selected to be densified as Phase 1. After completion of Phase 1, the intermediate drop points could be densified as Phase 2. Some projects use only one phase; others have been undertaken with as many as five phases.

1.7.2 Vibro-compaction

This glossary describes terminology unique to vibro-compaction.

Vibro-compaction – A ground modification technique that uses specially designed probe-type, depth vibrators for in situ densification of loose sands and gravels.

Vibro-diplacement – Refers to the dry, top or bottom feed process; almost no in situ soil appears at the surface, but is displaced by the backfill material.

Vibroflotation – Refers to compaction by means of a vibroprobe, sometimes called a vibroflot that densifies with predominantly horizontal vibrations. Vibroflotation is the original name for the process and is used synonymously with vibro-compaction.

Vibro-replacement – Refers to the wet, top-feed process in which jetting water is used to aid the penetration of the ground by the vibrator. Due to the jetting action, part of the in situ soil is washed to the surface. This soil is then replaced by backfill material.

1.8 Primary References

1.8.1 Deep Dynamic Compaction

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- GEC 1. (1995). *Dynamic Compaction*. Author: Lukas, R., FHWA SA-95-037, Federal Highway Administration, U.S. DOT, Washington, D.C., 97p.
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1.8.2 Vibro-Compaction

- Kirsch, K. and Kirsch, F. (2010). *Ground Improvement by Deep Vibratory Methods*. Spon Press, 189p.
- Massarsch, K.R. and Fellenius, B.H. (2002). Vibratory Compaction of Coarse-Grained Soils. *Canadian Geotechnical Journal*, 39(3): pp. 695-709.

- Massarsch, K.R. and Fellenius, B.H. (2005). Deep Vibratory Compaction of Granular Soils. Chapter 19 in *Ground Improvement – Case Histories*, B. Indranatna and J. Chu, Editors, Elsevier Publishers, pp. 633-658.

2.0 DEEP DYNAMIC COMPACTION

A number of fundamental questions must be addressed before proceeding with an in-depth evaluation of deep dynamic compaction at a specific project site. These include the following:

- What are the typical applications?
- What are the advantages and disadvantages?
- What types of deposits can be improved?
- Are there limitations or environmental considerations?
- Are there alternate site improvement methods?
- Is it cost-effective compared to other alternatives?

The following sections provide answers and insight to these questions.

2.1 Feasibility Considerations

2.1.1 Applications

The primary purpose of deep dynamic compaction is to densify natural and fill deposits to increase bearing resistance, reduce settlement, minimize collapse potential of large voids or collapse-susceptible soils, and mitigate liquefaction potential so that engineered structures such as bridges, abutments, retaining structures, and embankments can be constructed safely and economically. Discussion of the uses of deep dynamic compaction is best split into the following:

- Densification of loose deposits
- Collapse of large voids and collapse-susceptible soils
- Related applications

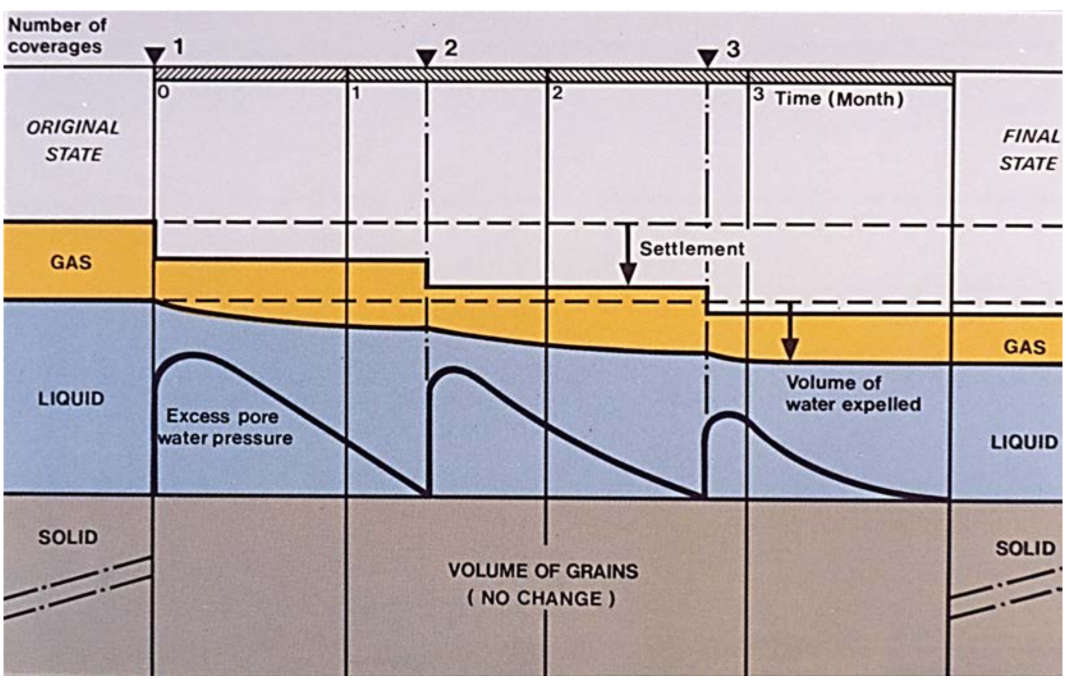
2.1.1.1 Densification of Loose Deposits

The primary use of deep dynamic compaction is to densify loose deposits so as to reduce the settlement that would otherwise occur under load application. Such deposits include natural soils and fill deposits of soils, manmade debris, byproducts, or any combination of these. However, deep dynamic compaction works best on dry granular materials including sands, gravels, ashes, mine spoils, and saturated, free-draining soils. The densification results from rearrangement of the material particles into a more compact state, resulting in a lower void ratio. In partially saturated soils, densification is similar to laboratory impact compaction by the proctor method. In saturated or nearly saturated fine sands and silts, excess pore water pressures develop on impact, which dislodges some of the point-to-point contacts between soil particles. Following dissipation of the pore water pressures, the soil grains rearrange into a denser state of packing at a lower water content.

In areas subject to seismic activity, saturated sand or silty sand deposits that are stable under static conditions may liquefy during a seismic event. Deep dynamic compaction induces high pore water pressures into these soils which liquefies the soils resulting in a denser state after dissipation of the pore water pressure, which in turn, makes them non-susceptible or less-susceptible to liquefaction from earthquakes.

Deep dynamic compaction has been used to improve fine-grained, cohesive soils, although such soils are more difficult to improve with DDC. In fine-grained soils with low saturation, DDC can be effective in collapsing the air void spaces. DDC works best in soils in which the water content is less than the plastic limit (PL). Generally, DDC is not recommended with soils in which the plasticity index (PI) of the soil is greater than 8 and which possess high degrees of saturation. In such soils the applied load is transferred to the pore water and the low permeability does not allow timely dissipation of the pore water pressures, limiting the settlement induced in such soils. Heave of fine-grained soils often occurs when subjected to DDC.

The soil behavior during deep dynamic compaction is illustrated in Figure 4-4.



Courtesy of Soletanche Bachy

Figure 4-4. Phase diagram model of soil behavior during deep dynamic compaction.

The soil deposit is characterized as a phase diagram consisting of three portions, i.e., solid materials, liquids, and gas. On the left side of Figure 4-4, the phase diagram represents the original state of the soil deposit. During the first phase of energy application, excess pore water pressure develops in the liquid portion and then dissipates with time. The time required for pore pressure dissipation is a function of the permeability of the soil mass and the distance to a drainage path. There is some induced ground settlement, which is represented by a lowering of the top of the phase diagram. This induced settlement results from expulsion of some of the gas and liquid for the voids, which causes a lower void ratio. The solid portion remains the same.

During the second phase of energy application, the pore water pressures temporarily increase during tamping, but this time, the magnitude is slightly less. At the end of the second phase, the excess pore water pressures have once again dissipated back to the original condition. The thickness of the liquid portion decreases slightly less than during Phase 1, and the gas portion is also slightly less than in Phase 1. The induced settlement increases beyond what occurred during Phase 1. The void ratio of the soil also has been lowered.

During Phase 3, excess pore water pressures are again generated, but to a lesser extent than during previous phases. Once again, some water is expelled as the pore water pressures dissipate and the induced settlement is slightly increased.

The final state of the soil mass is shown on the right side of Figure 4-4. The volume of the solids remains the same as the initial state, but the volume of the liquids and the gas decreases. The reduced void ratio results in reduced compressibility and increased strength of the deposit.

2.1.1.2 Collapse of Large Voids and Collapse-Susceptible Soils

Large voids may exist within the soil mass in either natural or fill deposits. Deep dynamic compaction is used to collapse the voids and provide a more uniform foundation that reduces differential settlement.

In karst formations, voids often develop within the soil deposit as a result of erosion into an underground karstic limestone cavern created by the dissolution of limestone or dolomite. The risk of unforeseen large settlement or soil collapse into the void can be reduced by deep dynamic compaction. For DDC to be effective, the void should be located within the effective depth of treatment as shown in Figure 4-1.

Man-made fill deposits from construction debris, solid waste, mine spoil, or mineral processing may have large voids created by the filling process. Voids can also be caused by buried vessels, such as drums or pipes. These voids could be collapsed by DDC, provided they are within the zone of treatment.

In mine spoil deposits, voids are frequently present in zones where slabs of rock or nested clusters of large boulders exist. During the placement of these materials, no effort is generally made to compact the deposits or to isolate the larger chunks within the soil matrix of the mine spoil. In time, these voids eventually fill in with erosion of the finer materials into the larger voids, which in turn results in ground subsidence.

Windblown soils such as loess and partially saturated soils deposited in arid environments often have a loose soil structure in which the soil grains are weakly cemented together. These soils are susceptible to collapse upon wetting as the cementitious bonds between the particles are dissolved and the soils collapse into a denser state of packing. Deep dynamic compaction can be used to overcome the cementitious bonds and move the particles into a more dense condition to reduce or eliminate settlement upon wetting.

2.1.1.3 Related Applications

In weak saturated soils relatively deep craters greater than 5 feet can develop. If these craters are filled with coarse granular material and supplemental energy applied, the granular material will be driven into the weak deposit. This type of improvement is strictly speaking not deep dynamic compaction and is called dynamic replacement. The dynamic compaction equipment is used to produce the improvement, so this procedure is a related form of ground improvement. The depth of improvement is generally less than about 10 to 13 feet.

2.1.2 Advantages and Potential Disadvantages

2.1.2.1 Advantages

The advantages of deep dynamic compaction include the following:

- Impacting the tamper into the soil serves as both a probing and a correcting tool. If there are weak ground conditions or large voids in local areas, the tamper will penetrate further into the ground than in adjacent areas, thereby causing large crater depths. This provides the field engineer with immediate feedback on ground response. A decision can then be made regarding further energy application in this area to correct the poor ground condition or, if the deposit will not compact upon energy application, to undercut and remove the poor ground. This probing aspect of deep dynamic compaction is important in heterogeneous deposits, such as old landfills, mine spoils, or in karst deposits where voids are present in local areas.

- Densification of the deposits can be observed as the work proceeds. If multiple passes are made over an area, each succeeding pass generally will result in an average crater depth less than that of the prior pass. This is an indication of the improvement from the resistance of the ground. Ground settlement readings are usually taken before and after each application of energy, and the amount of ground compression is an indicator of the degree of improvement achieved. Normally, ground compressions of 5 to 10 percent of the thickness corresponding to the depth of treatment, as predicted by Equation 4-1, occurs during densification. In extremely loose fills, such as recent landfills, the ground compression can be 20 to 25 percent.
- Deep dynamic compaction can be used at sites with a very heterogeneous mixture of deposits and at sites with gradation ranges from large boulders and broken concrete to silty soil particles. Deep dynamic compaction is effective in densifying all of these deposits with the same equipment. Furthermore, deposits that were formerly thought uncompactable, i.e., building rubble debris or decomposed sanitary landfills, can be compacted by this method.
- Densification usually results in a bearing stratum having a more uniform compressibility, which minimizes differential settlements. Weaker zones within the deposit undergo the most improvement, which eliminates zones of potentially high compressibility.
- Densification can be achieved below the water table in pervious and semi-pervious deposits, which eliminates costly dewatering and/or lateral bracing systems required for conventional excavation and replacement techniques.
- Except for the very heavy tampers and the high drop heights, non-specialty contractors can perform deep dynamic compaction on a local basis, making the cost for deep dynamic compaction very competitive. For the larger tampers and the higher drop heights, the equipment must be modified because cable and drum wear is higher than normal. In this case, specialty contractors are required to perform the work.
- Deep dynamic compaction can proceed during inclement weather conditions, including freezing weather or rain, provided precautions are taken to minimize water accumulation and frost penetration. Excess surface water should be removed by sloping the ground to shed water or by pumping. Deep frost penetration in fine grain deposits should be prevented by covering the surface with soil or straw removed immediately prior to deep dynamic compaction.

2.1.2.2 Potential Disadvantages

Potential disadvantages of deep dynamic compaction include the following:

- Deep dynamic compaction produces ground vibrations that can travel significant distances from the point of impact. In urban areas, this may require the use of lightweight tampers and low drop heights, as well as limiting deep dynamic compaction to areas well within the property lines. At some sites, shallow isolation trenches have been cut through the upper portion of the soil mass to reduce the transmission of energy off site.
- To prevent surface softening of the soil mass, as well as sticking of the tamper into the soil, the ground water table should be located more than 6 feet below ground surface. If the water table is higher than 6 feet, it may be necessary either to lower the water table by pumping or raise the grade by soil placement.
- At sites consisting of very loose deposits, such as recent landfills, it is frequently necessary to place a layer of granular material, such as gravel or crushed stone, at the surface to provide a working platform for equipment operation and to limit tamper penetration at impact. The surface layer also provides confinement for the underlying weak deposits. The cost of the granular fill can significantly add to the cost of the deep dynamic compaction operation.
- Lateral ground displacements of 1 to 3 inches have been measured at distances of about 20 feet from the point of impact of 30 to 60 kip tampers. Utilities or buried vessels within the zone of influence could be displaced or damaged.

2.1.3 Feasibility Evaluations

2.1.3.1 Geotechnical

The suitability of deep dynamic compaction is evaluated with the following soil parameters:

- Permeability of the soil mass, which can be inferred from soil classification
- Degree of saturation, which is related to the position of the water table
- Length of drainage path
- Soil stratigraphy, such as buried hard or weak layers

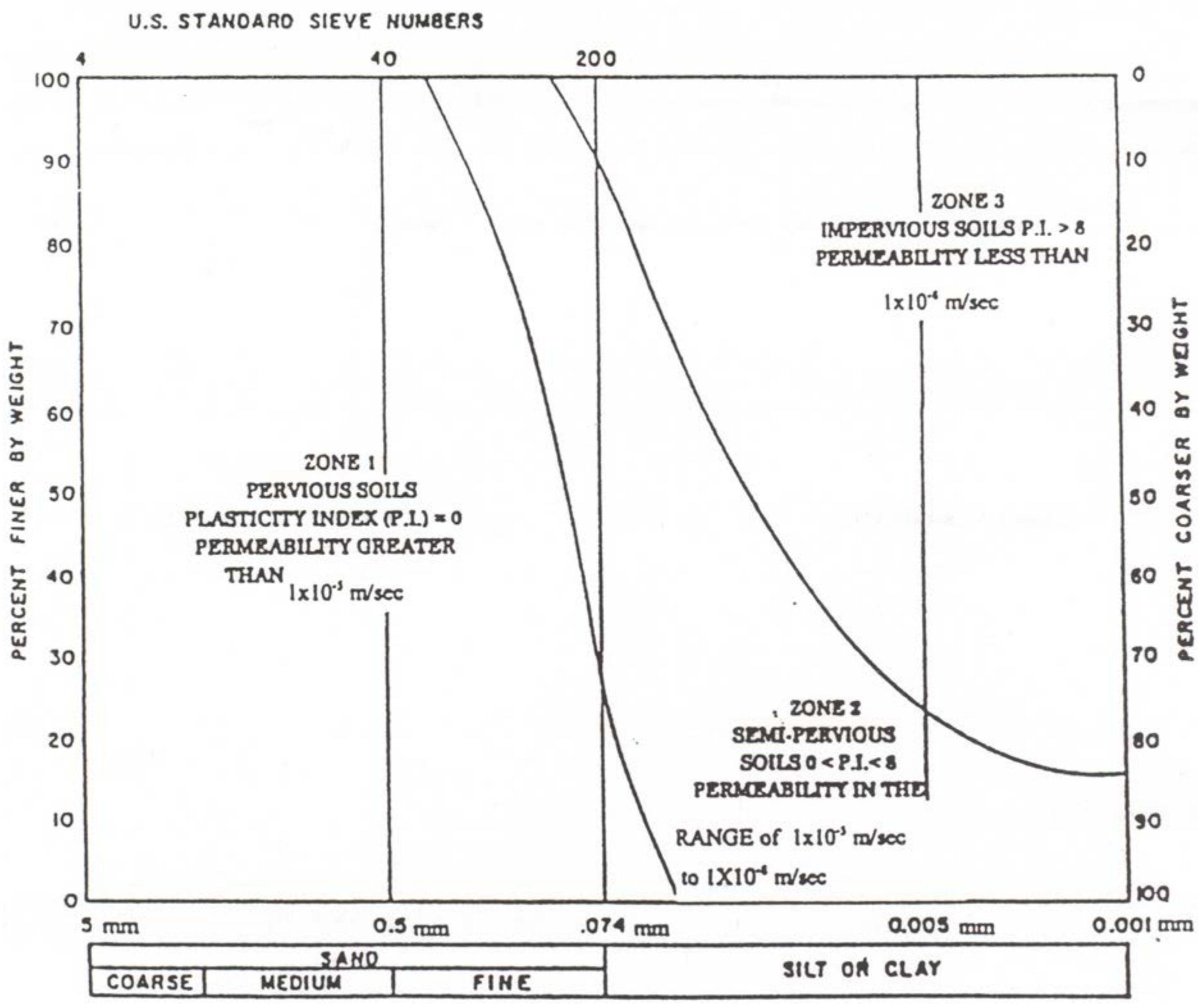
Deep dynamic compaction works best on deposits where the degree of saturation is low, the permeability of the soil mass is high, and the drainage is good. Deposits considered most appropriate for deep dynamic compaction are pervious granular soils. If these deposits are situated above the water table, densification is immediate, as the soil particles are compacted into a denser state. If these deposits are situated below the water table, the permeability of the soils is usually high, and excess pore water pressures generated by the impact of the tamper dissipate almost immediately and improvement is correspondingly immediate. Pervious granular deposits include natural sands and gravels and fill deposits consisting of building rubble, granular mine spoil deposits, industrial waste fills, such as certain types of slag, and decomposed refuse.

Deposits for which deep dynamic compaction is not appropriate include saturated clayey soils (either natural or fill). In saturated deposits, improvements cannot occur unless the water can be expelled from the voids. In clayey soils where the permeability is low, the excess pore water pressures generated during deep dynamic compaction require a lengthy period of time to dissipate, which renders deep dynamic compaction impractical for these deposits. Furthermore, the degree of improvement is generally minor in saturated clayey deposits. The upper portions of these deposits can be improved by dynamic replacement (see section 2.1.1.3).

Intermediate between the two extremes of granular pervious soils and saturated clay deposits is a third category of semi-pervious soils. Silts, clayey silts, and sandy silts are in this category. Deep dynamic compaction will work in these deposits, but because of the lower than desired permeability, the energy must be applied using multiple phases that allow excess pore water pressures to dissipate between energy applications. Sometimes the excess pore water pressure dissipation occurs over periods of days to weeks. At some project sites, wick drains have been used to shorten the drainage path, and thereby dissipate pore pressure.

Unfortunately, not all the influential soil parameters are determined as part of the preconstruction field exploration process, nor can they be measured with accuracy. An estimate of the field permeability can be obtained through slug tests performed in boreholes, but this is not undertaken for most projects. Instead, the permeability is generally inferred from soil classification or soil index tests. Some idea of the length of the drainage path can be obtained from examination of the soil boring logs, although thin sand seams within a fine-grained soil deposit could escape detection. Thus, drainage can only be estimated subjectively. In fine-grained soils, the degree of saturation can be estimated from laboratory tests such as unit weight, water content, and specific gravity. In coarse-grained soils, it can be assumed that the soils are partially saturated above the water table and fully saturated below.

A guide for estimating the suitability of deep dynamic compaction for various soil deposits is provided in Figure 4-5, based upon conventional index tests.



GEC 1 1995, After FHWA 1986

Figure 4-5. Grouping of soils for deep dynamic compaction.

Three categories of deposits shown in Figure 4-5 are summarized below.

- Zone 1 represents pervious soil deposits where deep dynamic compaction is best suited. The permeability of these deposits is generally greater that 10^{-5} m/s. Improvements are obtained regardless of whether these deposits are partially or fully saturated.
- Zone 3 represents impervious clayey deposits. The permeability of these soils is generally less than 10^{-8} m/s. Deep dynamic compaction is not recommended if these deposits are saturated or nearly saturated. Some improvement has been achieved in clayey fill deposits that are only partially saturated, provided the water content at the time of densification is at or below the plastic limit.
- Zone 2 represents intermediate soil deposit where improvements can be achieved, provided the energy is applied in multiple phases to allow complete pore pressure

dissipation between phases. The permeability of these deposits is generally between 10^{-5} to 10^{-8} m/s.

The suggested procedure for evaluating whether the soil deposits at any site are suitable for deep dynamic compaction is to first determine the grain size distribution of representative samples to determine where the gradation plots on Figure 4-5. If the gradation plots in Zone 1, proceed with the design and construction sequence for deep dynamic compaction. If the soils are in the range of Zones 2 or 3, additional testing, including water content and specific gravity, will help determine the degree of saturation. It would also be necessary to determine the position of the water table and to establish whether any pervious seams are present within the soil deposit. Additional field permeability testing should be considered since field permeability, not grain size gradation, dictates the suitability of the soil. If the testing confirms that the deposit is in Zone 3, consider alternate methods of ground improvement. If the deposits are classified as Zone 2, deep dynamic compaction will work, but the energy should be applied in phases.

On some projects, test sections are used to evaluate the effectiveness of deep dynamic compaction before construction. This is especially important when the soil stratigraphy is variable. Strata of hard or weak deposits within an otherwise uniform deposit can affect the depth and degree of improvement. Soil borings with Standard Penetration Tests (SPT), Cone Penetrometer Tests (CPT), or Pressuremeter Tests (PMT) are generally completed before and after energy application to measure soil improvement. Different levels of energy can be applied and evaluated. In fine-grained soils, piezometers can be installed to measure the magnitude of pore water pressures generated by deep dynamic compaction and the time for dissipation. Information gathered from a properly instrumented and monitored test section is very helpful in preparing more meaningful specifications that could lower construction costs and prevent construction delays.

2.1.3.2 Environmental Considerations

The key environmental considerations are noise, vibrations, and lateral movements, which can affect neighboring structures and people. Typical noise levels on an open site can reach 100 to 120 dB at 40 feet decreasing to less than 80 db at 80 feet (Dumas and Beaton 1992). Normally, the noise issue is only present during short periods during the lifting and dropping cycle. Ground vibrations can travel significant distances and may travel offsite affecting nearby property and structures. Detailed information regarding vibration analysis is included in this document in Section 2.1.4. Lateral ground displacement adjacent to craters may damage existing utilities and structures. During construction, airborne debris can cause damage or injury unless proper precautions are taken.

2.1.3.3 Site Conditions

- The use of deep dynamic compaction requires sufficient headroom for the cranes (equipment).
- Adjacent buildings, structures, utilities and property must be monitored for potential vibrations when using DDC.
- Vibration, noise and lateral movements.

Site conditions should always be considered when selecting a ground modification technology. Site topography and in situ soil conditions can have a considerable effect on the economics of a DDC solution. The site investigation should establish the site soils, whether clay, silt, sand or gravel; soil densities and water contents; the groundwater level; and the presence of overhead wires, buried utilities, or nearby structures. Some of the specific site and soil conditions that affect the economics or feasibility are listed below:

- *Uneven and unstable working surface.* The cranes for DDC need a level and stable working surface. If the ground surface is uneven, a working layer can be placed on which the cranes can operate. This layer is best comprised of free-draining materials. The weight of the cranes must be accommodated by the working platform.
- *Headroom*. To the large sized cranes used for DDC, sufficient headroom must be available for the operation to proceed. Overhead wires can possibly present a logistic problem. Overhead lines must be de-energized to avoid possibilities of accidental impact or arcing of current from the lines to the mast.
- *Obstructions above the compressible layer.* The key issue with obstructions such as concrete, rock, rubble, slag, brick, wood, riprap, stone, debris, rubbish, or trash near the surface is the reduction in energy reaching the compressible layer. If these materials are present in the upper reaches, DDC will help to densify such materials, but the energy transmission to the materials below will be reduced and additional drops may be necessary to achieve the desired compaction.
- *Depth.* The soil is improved to depths between 10 to 35 feet. If deeper improvement is necessary, deep dynamic compaction in combination with other systems, such as grouting and stone columns, should be considered.

Should any of the above site conditions be encountered, it would be advisable to contact specialty contractors experienced in DDC in order to determine the magnitude of difficulty.

2.1.4 Limitations

Whenever a tamper impacts the ground, vibrations are transmitted through the subsurface with diminishing intensity, as the distance from the point of impact increases. If deep dynamic compaction is undertaken close to property lines, ground vibrations transmitted off site should be considered.

There are no general regulations of construction and industrial vibrations. Based on research related to blasting, the U.S. Bureau of Mines found that building damage is related to particle velocity (Siskind et al. 1980). The Bureau of Mines developed Figure 4-6 based on experiences with damage measurements made in residential construction from blast-induced vibrations.

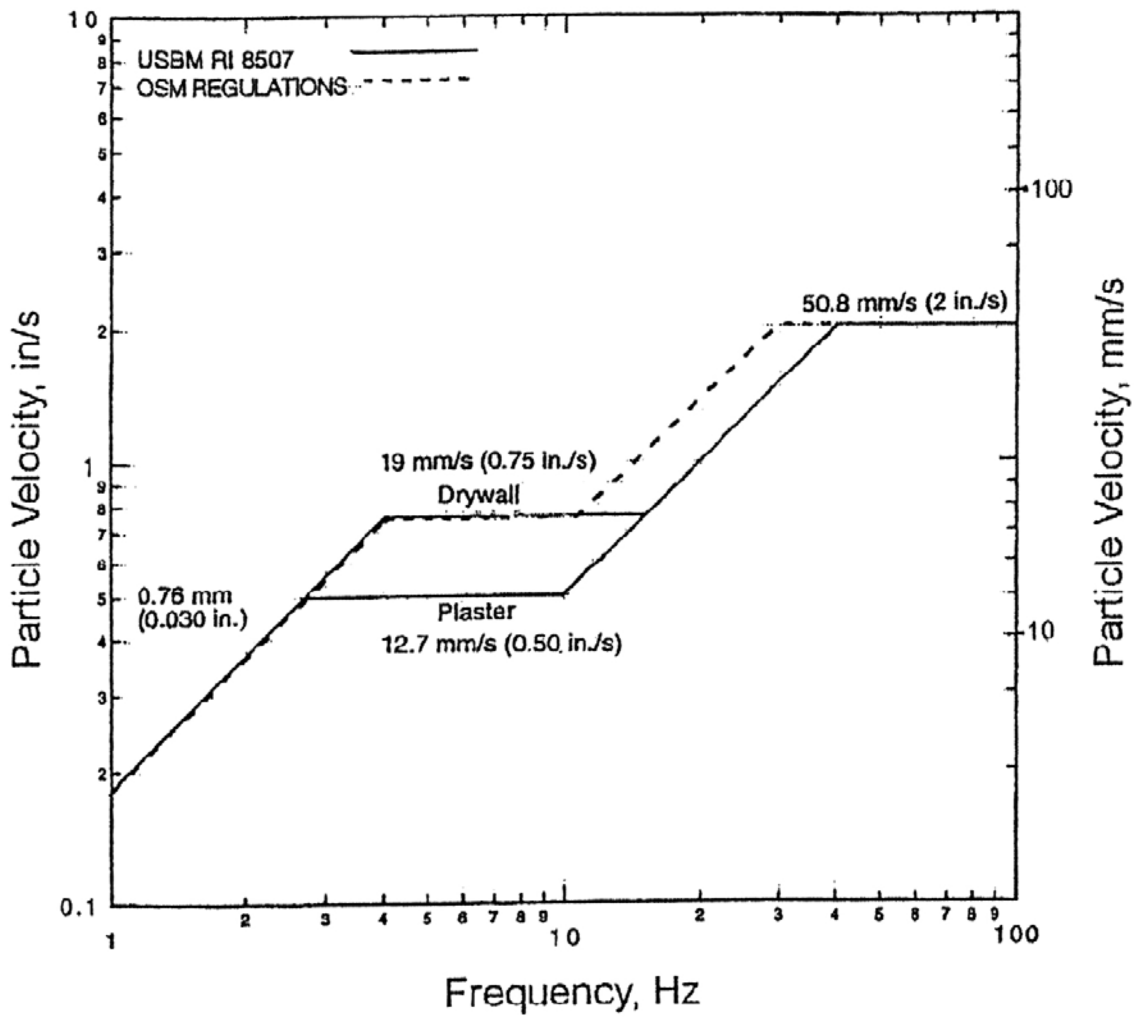


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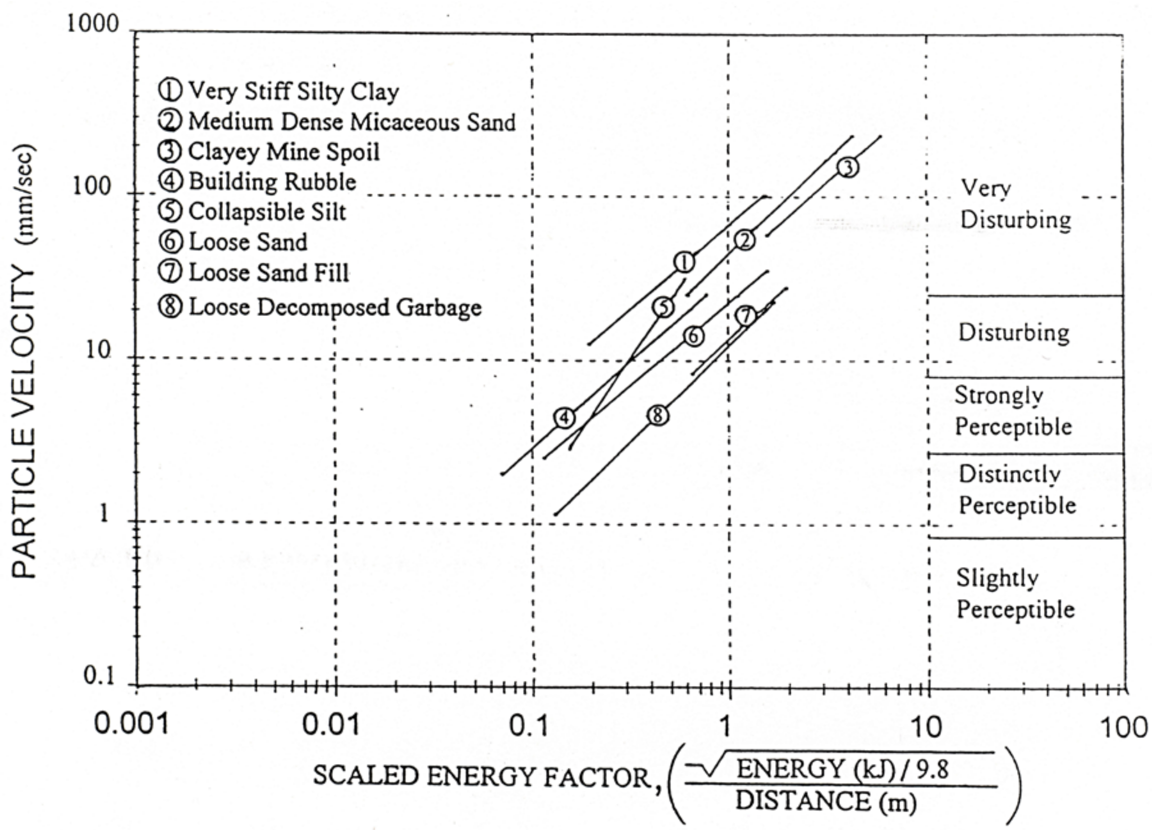
Figure 4-6. U.S. Bureau of Mines safe level of blasting vibrations for houses and Office of Surface Mining regulations.

The limiting particle velocity depends upon the frequency of the wave form. Normally, deep dynamic compaction results in frequencies of 5 to 12 Hz. Using Figure 4-6 as a guide, this would limit peak particle velocities to values of ½ inch/sec for older residences with plaster walls and ¾ inch/sec for more modern constructions with drywall. Peak particle velocities that exceed the values given in Figure 4-6 do not mean that damage will occur. Rather, these values are the lower threshold beyond which cracking of the plaster or drywall may occur.

Data generated by the U.S. Bureau of Mines indicate that minor damage occurs when the particle velocity exceeds 2 inch/sec, and major damage occurs when the particle velocity exceeds about 7.6 inch/sec. Thus, keeping the particle velocity less than about ½ to ¾ inch/sec should be a reasonably conservative value to minimize damage.

Normally, the ground vibrations are measured with a seismograph at the time of construction. The readings are taken on the ground adjacent to nearby structures. However, before starting deep dynamic compaction operations, it is necessary to predict the particle velocity of ground vibrations, because this may affect the level of energy application in close proximity to existing facilities. For planning purposes, Figure 4-7 can be used.



FHWA 1986

Figure 4-7. Scaled energy factor versus particle velocity.

The square root of the energy of a single drop (drop height times the mass of tamper) divided by the distance from the point of impact is used to calculate the scaled energy factor. This value, and the type of soil deposit that most closely resembles the soil being densified, can then be used to predict the particle velocity. The basis of Figure 4-7 is actual readings obtained at specific sites.

If the predicted particle velocity is higher than desired, it will be necessary either to reduce the energy or increase the distance between the point of impact and the adjacent facility. Either would reduce the scaled energy factor. At some sites, trenches have been dug along the property line to reduce the particle velocity. This was found to be partially helpful in reducing the surface waves that travel off site. The effectiveness of the trenches can be established at the time of construction from vibration readings taken on the near and far side of the trench following impact of the tamper.

Even though damage may not occur, ground vibrations will still be felt by humans; this can be annoying and lead to complaints. The relative response by humans is shown on the right side of Figure 4-7. Ground vibrations would be disturbing to people in the range of ½ to ¾ inch/sec (13to 19 mm/sec). Some education on the part of the owner or contractor will be necessary to reduce the fears of the adjacent property owners if the ground vibrations are in this range.

Buried utilities tolerate higher vibration levels without damage than do buildings because the utilities are surrounded in a soil mass. The utility moves with the soil mass and remains confined. Water mains and water pipes have sustained particle velocities of 3 inch/sec without damage.

Deep dynamic compaction also causes lateral ground shifting. Measurements taken for three different soil types, with inclinometers located at 3 m and 6 m (10 to 20 feet) from the point of impact of tampers in the range of 15-29 Mg (33-64 kips) are presented in Figures 4-8 and 4-9.

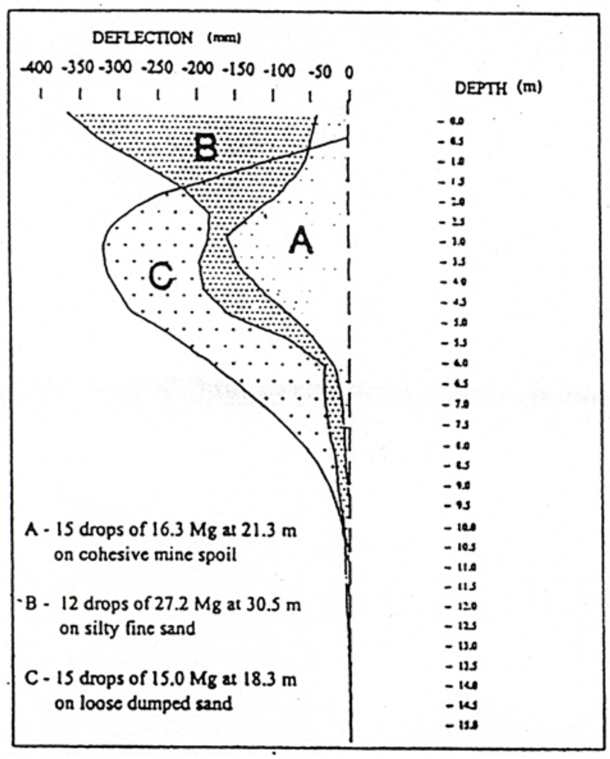


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Figure 4-8. Lateral movements at 3 m (10 feet) from drop point.

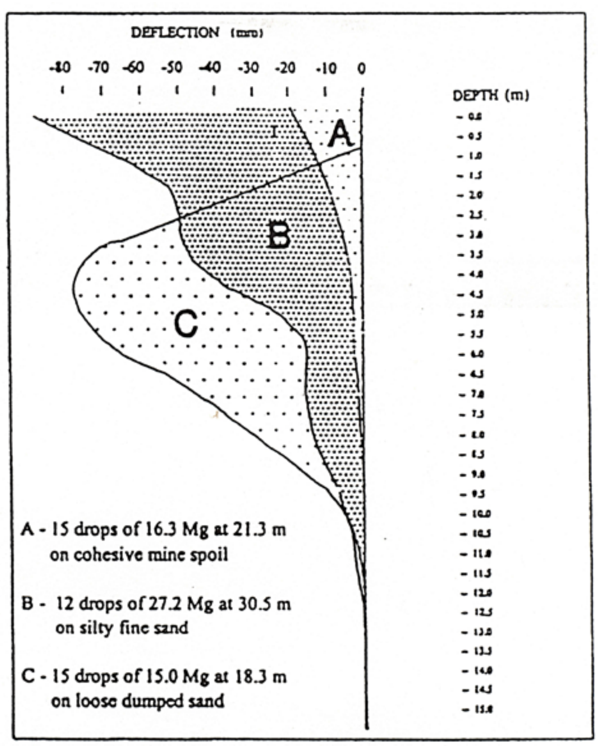

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Figure 4-9. Lateral movements at 6 m (20 feet) from drop point.

At a distance of 3.0 m (10 feet) from the point of impact, lateral deformations as much as 254 to 305 mm (10 to 12 inches) were observed, while at 6 m (20 feet), lateral deformations ranging from 13 to 76 mm (½ to 3 inches) were reported. These are permanent deformations that occur as the ground is displaced laterally following impact.

Based upon these measurements and observations from other sites, it appears that deep dynamic compaction operations with tampers ranging from 15-30 Mg (33-66 kips) should not be undertaken within about 6 to 7 m (23 to 26 feet) of any buried structures, if movement could cause damage to these structures. This could include a utility or a shallow structure foundation.

Occasionally, debris may become airborne following the impact as the tamper strikes the ground. This is likely to occur in dry soil deposits and those that contain larger objects, such as cobbles, boulders, or even landfill sites containing bricks and bottles. To avoid being hit with flying debris, a safe working distance from the point of impact should be delineated.

Where deep dynamic compaction is to be undertaken immediately adjacent to a street or another facility, it may be necessary to erect a shield or barrier to deflect flying particles.

Safety and cost issues arising from rapid cable wear must be addressed by the contractor. The solution may include the use of swivels, the reduction of cable lengths, use of a tag line, use of non-rotating surface cables, placement of rubber tires over the tamper, or other specialized equipment modifications or features (FHWA 1986).

There is a depth limitation on improvement. The heaviest tamper that can be lifted with conventional equipment is about 40 kips with a drop height of 75 to 90 feet. This will result in a maximum improvement depth of about 36 feet. If deeper improvement is necessary, deep dynamic compaction in combination with other systems, such as grouting and stone columns, should be considered.

2.2 Construction and Materials

2.2.1 Overview

Conventional lifting cranes are used for deep dynamic compaction projects where the tamper size is less than about 40 kips. Tampers are sometimes built especially for deep dynamic compaction, while other tampers have been fashioned from steel ingots or other sources, such as a bank vault door.

Associated pieces of equipment include a front-end dozer for leveling the ground after the craters are formed. Imported granular material may also be required to provide a firm working mat across the site.

Because of the conventional and readily available equipment required for most deep dynamic compaction projects, many general, demolition, or excavation contractors will perform deep dynamic compaction when properly guided and monitored.

2.2.2 Tampers

Deep dynamic compaction generally is performed with tampers ranging from 11 to 60 kips. The lighter tampers are used where the thickness of the deposit is relatively thin, such as 10 feet, and the heaviest tampers are used where the deposit is about 30 to 40 feet thick.

The tampers must be very rugged because high stresses are induced in the tamper when it strikes the ground. Most tampers are constructed of solid steel, but some have a steel base plate and steel sidewalls, with the interior filled with concrete. Tampers constructed solely of concrete have a relatively short life.

The tampers should have a flat base and can be either square, round, or hexagonal. Generally, the tamper rotates as it is lifted because the cable unwinds due to the heavy load. Therefore, a round tamper will always hit on the same imprint. Square tampers have also been used, but a rounded crater pattern develops. Sometimes guy wires are used to keep the tamper from rotating as it is lifted and dropped.

The contact pressure at the base of the tamper should be 5 to 10 psi. This pressure is obtained by dividing the weight of the tamper by the area of the base. A significantly higher tamper-pressure could cause penetration into the ground without densification. Pressures less than about (5 psi will distribute the energy over too wide an area to cause deep densification.

Photographs of 12, 30, 31, and 70 kip tampers are shown in Figures 4-10 through 4-13, respectively.

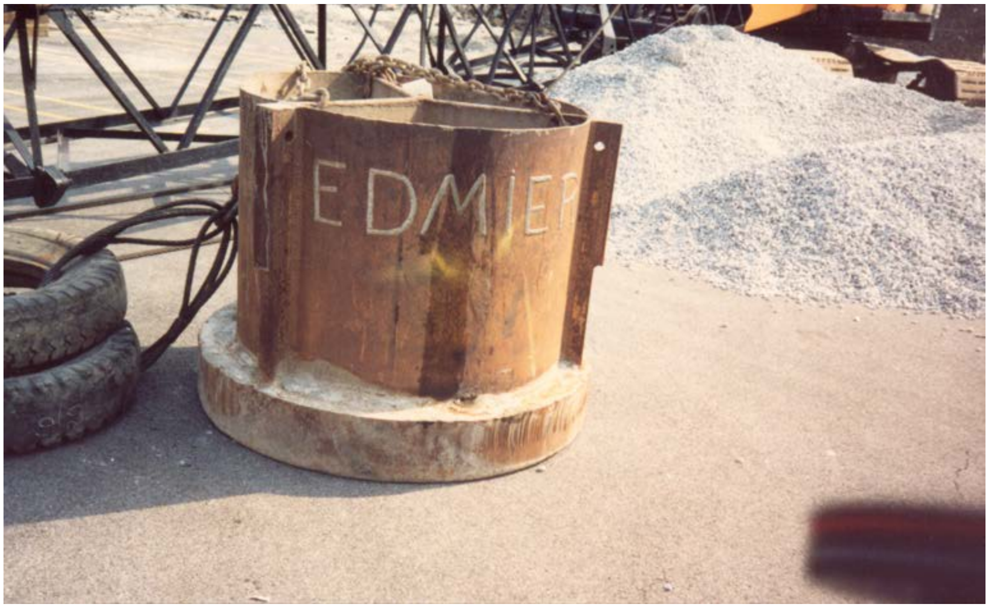


Figure 4-10. 12 kip tamper.

Figure 4-11. 30 kip tamper.

Figure 4-12. 31 kip tamper.

Figure 4-13. 70 kip tamper.

The 12 kip tamper has a round 6 inch thick steel base plate and a 1 inch thick steel cylinder that is filled with concrete. The 30 kip tamper also has a large base plate with steel plates stacked on top to make up the remaining portion of the tamper. The plates are welded and bolted together. The 70 kip tamper is solid steel.

Tampers with a low contact pressure are sometimes used to provide surface compaction after the deep densification is finished. The contact pressure at the base of these tampers is typically 1.5 to 5 psi. A tamper used for the ironing pass is shown in Figure 4-14. On some projects, surface densification is achieved with either conventional compaction equipment or by proofrolling with a fully loaded dump truck.

Figure 4-14. Tamper with low contact pressure for ironing pass.

2.2.3 Lifting Equipment

To deliver the maximum amount of energy to the ground, the tampers are lifted and dropped, using a single cable and a drum on the lifting equipment, which has a free spool. The only losses that occur from this type of equipment are friction in the free-spool drum, friction of the cable over the upper sheave of the boom, and some air resistance at the base of the tampers. Typical pieces of equipment used to raise and drop the tamper are shown in Figures 4-15 through 4-17.

Figure 4-15. Lifting crane of 70 kips rated capacity used with 12 kip tamper.

Figure 4-16. Dragline-type lifting crane with 1¼-inch cable for use with 40 kip tamper.

Figure 4-17. Crane with special lifting drum attached to rear of crane for use with 64 kip tamper.

There are also specialized pieces of equipment that will raise the tamper with multiple-part lines. Then a release mechanism allows the tamper to free fall. This also is an acceptable method for applying the energy.

Conventional lifting cranes generally are used for tampers in the size range of 10 to 40 kips. The type of crane and cable size required to repeatedly raise and drop tampers of different sizes are listed in Table 4-1.

Table 4-1. Equipment Requirements for Different Sized Tampers

Tamper Weight	Crawler Crane Size	Cable Size
8 to 16 kips	80 to 100 kips	¾ to ⅞ inch
16 to 28 kips	100 to 200 kips	⅞ to1 inch
30 to 36 kips	200 to 250 kips	1 to 1⅛ inch
36 to 50 kips	300 to 350 kips	1¼ to 1½ inch

For tamper weights in the range of 35 to 50 kips, the conventional lifting equipment must be modified to prevent breakdowns. When the tamper is released, a rocking motion causes a stress in the shaft connecting the cab to the tracks. To reduce this stress, spuds can be placed on the rigs to reduce rocking. In addition, the lifting drums are enlarged and made thicker to withstand the forces.

Specialized lifting equipment has been devised for tampers 50-60 kips. A crane developed for large-sized tampers is shown in Figure 4-18.

Figure 4-18. Lifting crane built only for deep dynamic compaction with 5.0-foot diameter lifting hoist and single line-rated capacity of 50 ton at line speeds of 85 feet/min used with 32 ton tampers.

The lifting drum is approximately 5 feet in diameter. The upper carriage of the crane is fixed to the tracks, so the crane cannot rotate in a horizontal direction. This reduces damage to the connection between the cab and the tracks but makes maneuvering around the site more difficult.

A crane using multiple part lines to lift a 64 kip tamper, that is then allowed to free fall and impact the ground, is shown in Figure 4-19.

Figure 4-19. Multiple part line used to lift 64 kip tamper (below grade in photo).

2.2.4 Surface Stabilizing Layer

Where soft ground conditions prevail, it may be necessary to place a surface stabilizing layer to allow travel of the deep dynamic compaction equipment across the site, as well as to reduce penetration of the tamper into the ground. Soft deposits would include fairly recent landfills with a thin cover, or a mine spoil deposit that has weathered to a softer clay consistency at the surface.

The stabilizing layer usually consists of a granular material with a typical gradation range from 6 inch maximum size, down to sand size. The thickness of these layers depends upon the stability of the surface deposits, but thicknesses ranging from 1 to 3 feet have been used successfully. Deep dynamic compaction equipment is shown in Figure 4-20 working on a site where crushed rock was placed to a depth of 3 feet over a landfill.

Photo courtesy Bob Lukas, Ground Engineering Consultants, Inc.

Figure 4-20. Deep dynamic compaction over a 3-foot layer of crushed rock placed on a landfill.

At sites where the deposits are more stable (i.e., building rubble deposits, loose granular outwash deposits, or old decomposed landfills that are elevated above the water table), surface stabilizing layers are not needed. Because the stabilizing layer can cost as much as the deep dynamic compaction, it is used only where absolutely necessary.

2.2.5 Construction Sequencing

Deep dynamic compaction is generally undertaken on a grid pattern throughout the entire project area and extends beyond the limits of the project for a distance equal to the thickness of the weak deposit being densified. Energy can be applied using single or multiple phases, as single or multiple passes.

A phase is the application of the energy in a specific pattern. For instance, initial energy applications undertaken on 25 foot centers across the area could be labeled Phase 1. Phase 2 could be the application of energy midway between initial grid points. A single phase of energy application is most often used on projects where pore pressures dissipate rapidly and the energy can be applied on grid point locations immediately adjacent to a grid point that has just been densified. Multiple phases are used on projects where pore pressures rise and take some time to dissipate; therefore, it is more effective to apply energy on an intermittent grid pattern.

A pass is the application of energy in increments at each specific drop point location. For instance, if the plan is to impart 12 drops at a specific grid point location, but only 3 drops can be applied before the crater depths become excessive or ground heaving occurs, the first 3 drops would be called the first pass. After the first pass is completed, pore water pressures are allowed to dissipate, and the craters are filled. As additional drops are applied, they would be called the second pass. In fine-grained deposits, it is sometimes necessary to use 3 or 4 passes, whereas in the more pervious deposits, only 1 pass is needed.

While most projects are dynamically compacted on a grid pattern, some projects require additional energy application at specific locations. At the footing locations, for instance, additional energy can be applied. On projects where there are karst formations, the induced settlement may be larger in some areas, which indicates the presence of voids; additional energy can be applied at these locations.

2.3 Design

2.3.1 Design Considerations

If the preliminary design considerations discussed in Section 2 indicate that deep dynamic compaction will be appropriate on a project, the next step is to prepare a more specific or detailed plan for the deep dynamic compaction procedures to be used. This plan would include the following:

- Determine the project performance requirements for the completed structure
- Select the tamper mass and drop height to correspond to the required depth of improvement
- Estimate the degree of improvement that will result from deep dynamic compaction

- Determine the applied energy to be used over the project site to produce the improvement

The information presented in this chapter is a brief summary of the design concepts. The FHWA GEC 1 (1995) provides additional details.

2.3.2 Performance Requirements

Deep dynamic compaction densifies the soil mass and this, in turn, improves soil shear strength and reduces compressibility. The minimum property values required for adequate performance of the new facility should be determined using conventional analysis, which is usually based upon Standard Penetration Test (SPT), Cone Penetration Test (CPT), or Pressure Meter Test (PMT) results. The required property values can then be compared with the estimated property improvements, following densification by methods outlined in this chapter. On this basis it can be determined whether deep dynamic compaction is capable of producing the desired effect.

As an example, if a roadway embankment is to be constructed over weak ground, one concern is settlement of the embankment. Most embankments can withstand up to 6 inches of settlement without detrimental performance of the pavement system, provided the settlement is reasonably uniform, doesn't occur next to a pile-supported structure, and occurs very slowly (Holtz 1975). Based upon the estimated properties of the soil following densification, a settlement prediction can be made for the height of embankment to be constructed, to determine if the settlement will be less than 6 inches. If so, deep dynamic compaction would satisfy the project requirements. Conversely, if the predicted settlement using conventional analysis would exceed what the embankment can tolerate, other methods of supporting the embankment should be considered.

Where bearing capacity of the foundation soil is a concern, a similar procedure could be followed. Estimated properties based on procedures given in this chapter could be used in a conventional analysis to determine if the bearing capacity has increased sufficiently to prevent failure.

If the purpose of densification is to prevent liquefaction, conventional analyses are generally based upon SPT tests or CPT tests. The minimum SPT or CPT value to prevent liquefaction is determined by this analysis. The estimated SPT or CPT value following densification can be determined from the procedures in this chapter. If the minimum SPT and CPT values are attainable, then deep dynamic compaction would be one suitable method of ground improvement.

The procedures in this chapter for estimating improvement are conducted before the project begins, but verification of the improvement by additional field testing is required.

In uncontrolled fill deposits, there is always the possibility of an extremely loose pocket of soil in an otherwise medium dense fill deposit. This loose pocket of soil may not have been encountered in any of the initial borings, but if the engineer is aware that the site consists of uncontrolled fill, deep dynamic compaction could reduce the risk of differential settlement from these unforeseen pockets of loose ground. Since deep dynamic compaction is undertaken on a grid pattern throughout a site, the presence of the loose pockets will show up during the compaction process. Additional energy can be applied to densify these loose fills, or some of the loose deposits can be undercut and replaced with a granular material that is then dynamically compacted.

2.3.3 Design Procedure

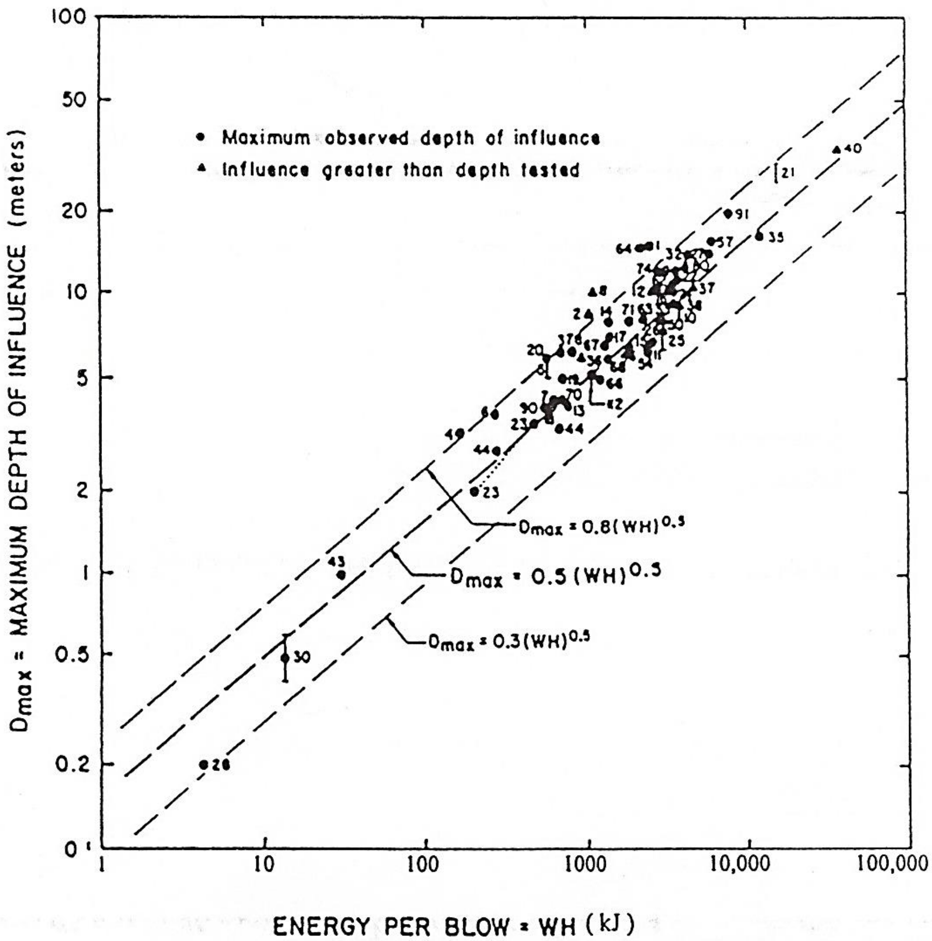
2.3.3.1 Depth and Degree of Improvement

Depth of improvement is a function of a number of variables, including the mass of the tamper, the drop height, the soil type, and the average energy applied. All factors are accounted for in the following relation:

$$D = n(WH)^{1/2} \qquad \text{[Eq. 4-1]}$$

where,

D	=	depth of improvement in meters
n	=	empirical coefficient that is approximately 0.5 for most soils. As shown in Figure 4-21, n varies between 0.3 and 0.8. A value of 0.4 is typically used for landfills.
W	=	mass of tamper in Megagrams (metric tonnes = 1.10 US ton)
H	=	drop height in meters (1 meter = 3.28 feet)

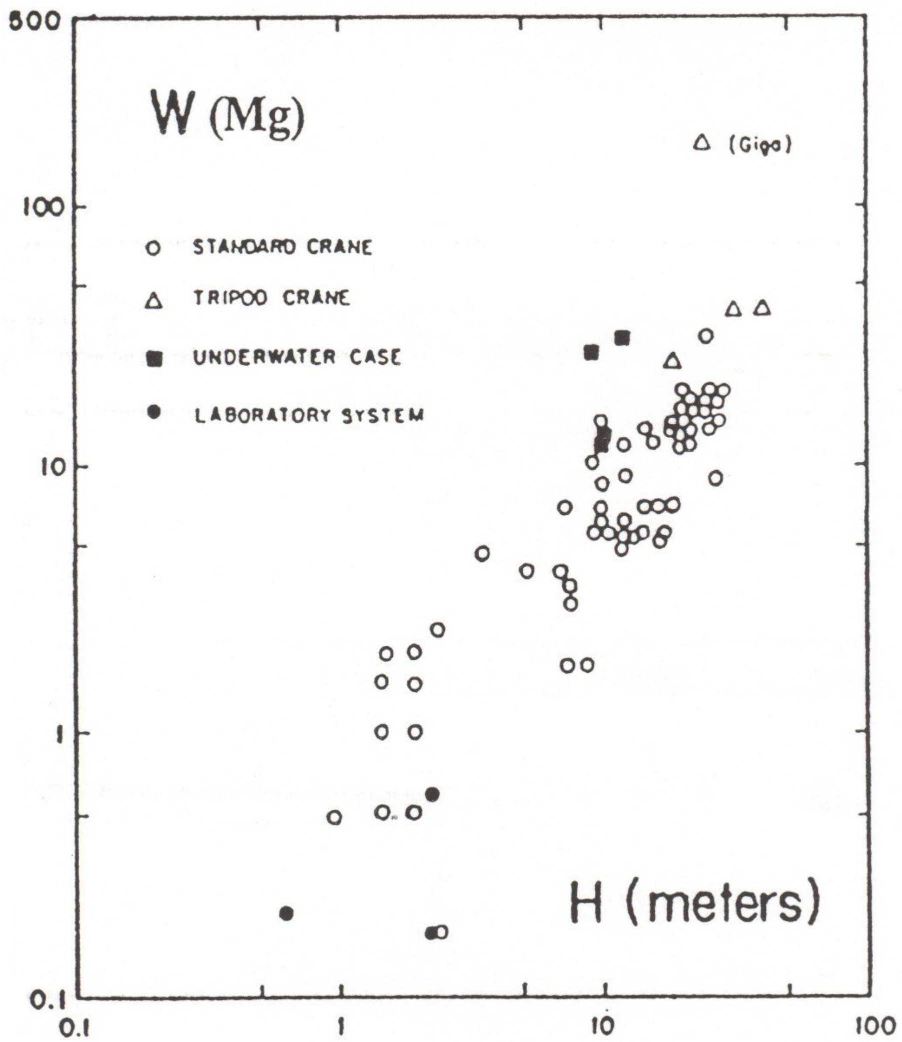


Mayne et al. 1984

Figure 4-21. Trend between apparent maximum depth of influence and energy per drop.

Other factors affecting the depth of improvement include the presence of soft energy absorbing layers, such as low strength clay or organic deposits, and/or hard layers at the surface that do not allow the energy to be transmitted to greater depths. A more thorough discussion of factors influencing the depth of improvement is presented in FHWA (1986) and Han (2015).

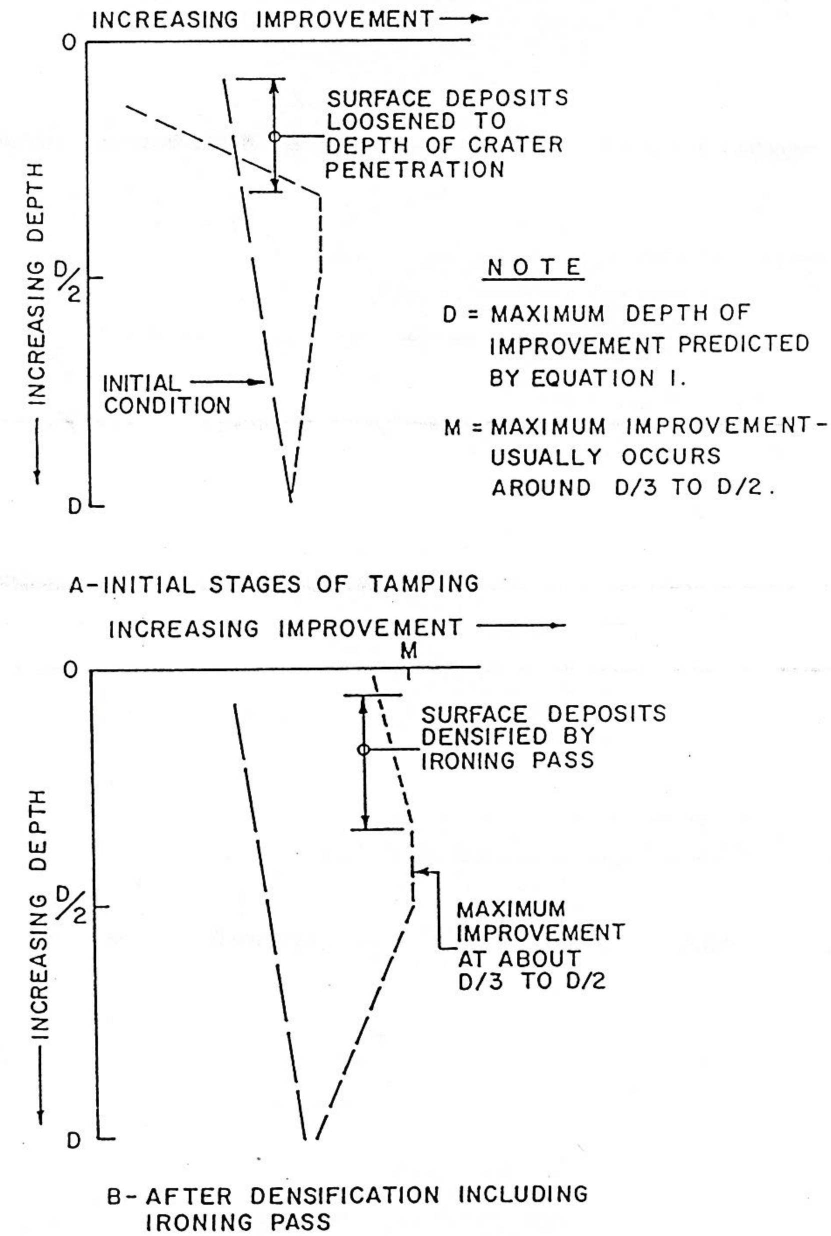
Various combinations of W and H can be used in Equation 4-1, depending upon equipment availability. The relationship between drop height and tamper mass that has been used on numerous projects is summarized in Figure 4-22. This figure can be used as a starting point with adjustments in W and H made after a contractor is selected.



1 meter = 3.28 feet; 1 Mg = 2.2 kips
Mayne et al. 1984

Figure 4-22. Relationship between tamper mass and drop height.

The amount of improvement resulting from deep dynamic compaction is generally measured by conventional in situ testing techniques such as SPT, CPT, or PMT. Test values obtained after deep dynamic compaction are compared with initial values before deep dynamic compaction to monitor the improvement. The greatest amount of improvement is generally near the upper portion of the soil layer densified and then decreasing with depth. The variation of improvement with depth is illustrated in Figure 4-23.



GEC 1 1995, After FHWA 1986

Figure 4-23. Variations in improvements with depth during deep dynamic compaction.

The degree of improvement achieved is primarily a function of the average energy applied at the ground surface. In general, the greater the amount of energy, the greater the degree of improvement. However, there are limitations to the maximum SPT, CPT, or PMT values that can be reached.

Based upon a review of a number of project sites, upper-bound test values of SPT, CPT, and PMT tests are shown in Table 4-2 (FHWA 1986). These indicate maximum values of improvement at depths of D/3 to D/2. Above and below this depth range, the test values would be less. At project sites where these test values were measured, the average energy applied was typically on the order of 41,300-62,000 ft-lbf/ft^2. If a lesser amount of energy is applied, or if there is some other complicating factor, such as energy absorbing layers or a hard surface crust that does not allow the energy to penetrate deeper, the degree of improvement could be significantly less than that shown in Table 4-2.

Table 4-2. Upper Bound Test Values after Deep Dynamic Compaction

Soil Type	Standard Penetration Resistance (blows/12 inches)	Static Cone Tip Resistance (tsf)	Pressuremeter Limit Pressure (tsf)
Pervious coarse-grained soil: • sands and gravel	30 to 50	200 to 300	20 to 25
Semipervious soil: • sandy silts • silts and clayey silts	25 to 35 20 to 35	135 to 177 104 to 135	14.6 to 20 10.4 to 14.6
Partially saturated impervious deposits: • clay fill and mine spoil	20 to 40*	N/A	14.6 to 20
Landfills	15 to 40*	N/A	5.2 to 10.4

* Higher test values may occur because of large particles in the soil mass.
Source: FHWA 1986

The upper-bound test values given in Table 4-2 should be used only for estimating the maximum degree of improvement that is possible at a project site. The actual improvement may be less and should be measured by field tests, such as SPT, CPT, or PMT, after deep dynamic compaction. Some testing should be conducted while the dynamic compaction equipment is at the site so additional energy application can be applied if the degree of improvement has not reached the desired value. However, it has been found that there is a delayed improvement with time following deep dynamic compaction, after all excess pore water pressures have dissipated. Lukas (1997) reported on 5 projects where improvements in soil properties were measured years after deep dynamic compaction. The improvements in standard penetration or pressuremeter tests ranged from 15 to 200 percent. Schmertmann (1991) stated that improvements of 50 to 100 percent in many soil properties can occur.

The reason for the strength gain with time is not fully known. This phenomenon has been attributed by various authors to cementation between soil grains, secondary consolidation, and aging. This delayed strength gain occurs mostly in fine-grained soils, but has also been measured in granular soils. Thus, if the tests taken at the end of deep dynamic compaction are only borderline acceptable, additional testing could be conducted at a later date to confirm the improvement that occurs in soil properties with time.

A detailed discussion of the reasons for delayed improvements from site improvement is discussed in Schmertmann (1991).

2.3.3.2 Energy Requirements

Deep dynamic compaction is generally undertaken on a grid pattern throughout the area. For this reason, it is convenient to express the applied energy in terms of average values. This average applied energy can be calculated on the basis of the following formula:

$$AE = \frac{(W)(H)(N)(P)}{(grid\ spacing)^2} \qquad \text{[Eq. 4-2]}$$

where,

AE	=	applied energy in tm/m^2 (9.76 tm/m^2 = 1 TSF)
N	=	number of drops at each specific drop point location
W	=	tamper mass in Mg (metric tonnes; 1 metric tonne = 1.10 US ton)
H	=	drop height in m (1 meter = 3.28 feet)
P	=	number of passes

Grid spacing in m (1 meter = 3.28 feet)

In using Equation 2, the following unit conversions may be useful, depending on units used:

1 metric tonne = 9.807 kN = 1 Mg

1 U.S. ton = 0.907 metric tonne

1 tonne-meter/m^2 = 9.807 kJ/m^2 = 0.1024 TSF

If different sized tampers and drop heights are used, the average applied energy would be the sum of all the levels of effort. For example, the high level of energy is generally applied with a heavy tamper and a high drop height. This is frequently followed with a low level of energy (called an ironing pass), using a smaller sized tamper and a lower drop height. The average applied energy would be the sum of the energy imparted to the site.

Most projects have been completed using energy levels ranging from approximately 21,000 to 62,000 ft-lbf/ft^2 (1 to 3 MJ/m^2). However, the energy can be varied for each project site depending upon the degree of improvement needed to satisfy the project requirements.

During the initial design or planning stages, an estimate of the required energy can be obtained using Table 4-3 in units of volume.

Table 4-3. Applied Energy Guidelines

Type of Deposit	Unit Applied Energy ft-lbf/ft^3 (kJ/m^3)	Percent Standard Proctor Energy
Pervious coarse-grained soil (Zone 1 of Figure 4-5)	4,100 to 5,200 (200 to 250)	33 to 41
Semipervious fine-grained soils (Zone 2 and clay fills above the water table and Zone 3 of Figure 4-5)	5,200 to 7,200 (250 to 350)	41 to 60
Landfills	12,400 to 22,700 (600 to 1100)	100 to 180

Note: Standard Proctor energy equals 12,400 ft-lbf/ft^3

This table incorporates some of the variables that influence the amount of energy required to achieve adequate improvement. This includes the thickness of the deposit, the type of soil being densified, and the initial relative density of the deposit.

The deposits are grouped into three broad categories of soils, with landfills requiring the greatest amount of energy and coarse-grained soils the least. The thickness of the deposit has been considered in Table 4-3 by listing the applied energy in terms of a unit volume. The average energy to be applied at the surface of the deposit can be obtained by multiplying the Unit Applied Energy by the thickness of the deposit. The initial relative density of the deposit is taken into account by showing a range in the amount of energy required. Within any particular soil type, the loosest deposits would require the higher level of suggested energy, and the denser deposits would require the lower level of suggested energy.

2.3.4 Example

An example helps illustrate the use of Table 4-3 for planning purposes. Consider a site that consists of a loose rubble fill with a thickness of 26 feet. The rubble fill can be classified as a pervious coarse-grained soil, typical of Zone 1. Because the deposit is in a loose condition, the upper bound of applied energy for this deposit of 5,200 ft-lbf/ft^2 would be used. Since the deposit is 26 feet thick, 41,000 ft-lbf/ft^2 of average applied energy is required. Substituting this number into Equation 4-2, and using the tamper mass and drop height from Equation 4-1, different combinations of drops, grid spacing, and passes can be evaluated.

The relationship between the number of drops, N, and the number of passes P, and the calculated applied energy is developed by Equation 4-2. The required mass of tamper, W, and drop height, H, is calculated from Equation 4-1. The grid spacing usually is selected as

1.5 to 2.5 times the diameter of the tamper. Equation 4-2 is solved for N *x* P. It is more efficient for the contractor to apply all the energy in one pass, than multiple passes. Using P=1, the number of drops at each grid point can be determined.

Multiple passes are used when not all the drops can be made at each grid point at one time, while the product of N and P remains the same whether there is a single pass or multiple passes.

The energy requirements shown in Table 4-3 can be related to standard proctor energy (ASTM D698), which is equal to 12,400 ft-lbf/ft^2. Pervious coarse-grained soils generally require about 33 to 41 percent of this energy to achieve adequate densification, and this is caused by some densification that occurs naturally in these deposits after they have been in place for a while. Conversely, landfill deposits require 100 to 180 percent of the standard proctor energy. Landfills are generally in an extremely loose condition and may even be underconsolidated, so a significantly higher level of energy is required to densify these formations.

Close monitoring of the field operations coupled with SPT, CPT, or PMT during deep dynamic compaction should be used to verify whether the desired improvement is reached.

2.4 Construction Specifications and Quality Assurance

This chapter presents a brief summary of methods that can be used in contracting for deep dynamic compaction, plus an overview of the monitoring that normally occurs during the field work. Complete construction specifications are detailed in FHWA GEC 1 (1995). Additional information on specifications can also be found in *GeoTechTools*.

2.4.1 Contracting Procedures

The designer will develop a set of drawings and specifications for the contracting package. This information is used by the construction personnel. It establishes the basis of payment, and details of the quality assurance program. Two contracting approaches are available in preparing the plans and specifications:

- A method approach
- A performance approach

With the method approach, the designer outlines the work plan that the contractor is to follow. Information is provided on the deep dynamic compaction process, including the size of tamper, the drop height, the average energy to be applied, whether the energy must be applied in single or multiple passes or phases, the need for a working mat, and all other facets of deep dynamic compaction that are necessary to achieve the proper improvement.

The method specification approach is more common as knowledge of deep dynamic compaction and the amount of energy to apply becomes better known among designers. The method specification also allows for general contractors to undertake some of the work.

With the performance approach, the contractor is responsible for achieving the desired result. The designer in this case specifies the required depth and degree of improvement and also provides information regarding the site and soil conditions. The contractor is free to select the size of the tamper, the drop height, and other specifics of deep dynamic compaction to achieve the requirements set forth by the designer.

Only experienced contractors should be allowed to bid work under a performance approach. There are approximately five to eight specialty contractors in the United States who specialize in deep dynamic compaction. Since these contractors will do a significant amount of engineering and planning of the deep dynamic compaction operation, the cost for deep dynamic compaction using this approach is generally higher than for projects where method specifications are used.

2.4.1.1 Method Specification

When preparing a method specification, the designer should include the following items in the plans and specifications:

- Tamper mass and size
- Drop height
- Grid spacing
- Applied energy
- Number of phases or passes
- Site preparation requirements
- Surface compaction after deep dynamic compaction
- Drawings of the working area
- Subsurface investigation data
- Test section design, if relevant

The owner or designer is responsible for the following:

- Monitoring during construction
- Borings and tests after deep dynamic compaction

The contractor is responsible for these items:

- Providing adequate equipment to complete the work in a timely manner
- Ensuring the safety of personnel and equipment
- Providing a work plan subject to approval by designer

2.4.1.2 Performance Specification

If a performance specification is prepared, the designer should specify the desired end product including these items:

- Minimum soil property value to be achieved and the method of verification
- Maximum permissible settlement
- Other objectives of site improvement
- Minimum contract qualification requirements

The owner or designer also provides the initial subsurface data and the lateral extent of the project site. Some site monitoring is also required, as it is in the interest of the owner to be aware of the details of the field operations, including any changes in the planned scope.

The contractor is required to meet the minimum specified final product and is responsible for the following:

- Proper equipment and a work plan
- Meeting the project deadline
- Safety
- Field monitoring
- Additional subsurface exploration, as required to properly prepare a deep dynamic compaction plan
- Verification of the end product

2.4.2 Instrumentation Monitoring and Construction Control

Regardless of whether deep dynamic compaction is performed under the method or the performance approach, it is essential to monitor the deep dynamic compaction operations.

During deep dynamic compaction, close observation of the procedures should be undertaken, because it is frequently necessary to adjust the field program. Reasonable and practical adjustments can only be made if good monitoring information is available. Monitoring during deep dynamic compaction includes these procedures:

- Where deep dynamic compaction is planned adjacent to structures, vibration readings should be taken to determine if ground vibrations pose a potential risk for the buildings. Building damage has been correlated to peak particle velocities as measured by seismograph, and there are charts available for determining permissible levels of vibrations.
- Measuring crater depths and adjacent ground surface heave at occasional drop point locations to determine the proper number of drops that can be applied at a location for maximum efficiency. As an example, the first few drops generally result in significant crater displacements without surrounding ground heave. After six or seven drops at one point, the crater depths may become significantly less for each drop, and some ground heave may occur. At some point, the additional crater volume produced by a single drop may be equaled by ground heave adjacent to the drop point location, which means that there is no longer any compaction occurring in the soil mass. Because there is merely a ground displacement at this time, application of additional energy would not produce densification.
- The average ground subsidence following deep dynamic compaction in an area should be measured. Settlement readings should be obtained on a grid basis over an area before and after deep dynamic compaction to determine the average induced subsidence. Generally, the induced subsidence ranges from 5 to 10 percent, except in landfills, where it could be higher.
- Field testing could be performed while the deep dynamic compaction is under way to confirm that the desired improvements are being reached. This could include soil borings with SPT, CPT, or PMT. At some sites, such as landfills, in situ soil testing is often meaningless and load tests are frequently made with settlement readings generally taken over a period of at least 1 week to measure performance. Field testing is only an indirect indicator of the strength and compressibility of the deposits shortly after deep dynamic compaction. The properties of some soils improve with time, which must be kept in mind when evaluating the results.

- Where there are fine-grained materials, such as silts or clayey silts, it may be helpful to install piezometers in the deposits below the water table. The purpose of this monitoring is to determine the magnitude and rate of dissipation in excess pore water pressure during deep dynamic compaction. This would determine the waiting period between drops applied at a location.
- In addition to the testing described above, qualified personnel should make general observations of the deep dynamic compaction operations. Adjustments, in the field, of the tamping program can be made depending upon these observations. For instance, where ground subsidence is much greater in one area than in the remaining areas, this could indicate an extremely loose pocket that requires additional tamping. Excessive ground heave in other areas might indicate soils that will not properly compact, which may require either a greater waiting period between densification or partial undercutting and replacement with a soil that will densify under deep dynamic compaction.

After the deep dynamic compaction is completed, conduct additional field explorations to confirm the degree and depth of improvement. This investigation is undertaken with soil borings along with SPT, CPT, or PMT. Because improvements in SPT, CPT, and PMT values have been observed 30 to 60 days following completion of deep dynamic compactions, it would be helpful if borings and field testing could be undertaken at that time.

On some projects, settlement plates and inclinometers have been installed following deep dynamic compaction to monitor the movement of the subsoils during construction of the embankment or structure. While not necessary, this practice does provide useful information to evaluate the effectiveness of the deep dynamic compaction.

For projects completed under a method type of specification, the owner and designer are responsible for monitoring operations. This would include providing field personnel during the deep dynamic compaction to confirm the depth and degree of improvement.

For projects completed under a performance type specification, the contractor is responsible for providing field monitoring and providing borings with SPT, CPT, or PMT after deep dynamic compaction to confirm the depth and degree of improvement. However, it is in the owner's interest to have field personnel on the site, because any changes to be made in the deep dynamic compaction plan would need to be mutually agreed upon between the owner and the contractor. The borings and field tests made after deep dynamic compaction should also be monitored to confirm that the improvement has been achieved.

2.5 Cost Information

2.5.1 Cost Components

The cost of deep dynamic compaction will vary as a function of the depth of improvement required. A greater depth of improvement requires a heavier tamper and a higher drop height. Therefore, the equipment to undertake this work also requires a much heavier lifting crane. For tampers up to 40 kips, costs are relatively predictable because conventional lifting equipment can be used to raise and drop the tamper repeatedly. A comparison of costs for deep dynamic compaction versus the size of tamper are presented in Table 4-4.

Table 4-4. Deep Dynamic Compaction Costs

Size of Tamper Required kips	Unit Cost Dollars/ft^2
4 to 8	1.10 to 1.50
8 to 16	1.50 to 2.00
16 to 20	2.00 to 3.00
20 to 90	Negotiated for each job

It should be noted that these costs include mobilization for large projects. On small projects, the mobilization costs should be added to the unit rates in the table. The unit costs do not include the cost of the quality assurance program or the cost of granular fill, if required, to fill craters or provide a surface stabilizing layer.

2.5.2 Cost Data

Costs for deep dynamic compaction vary considerably depending upon geographic location, type of deposit to be densified, and availability of local contractors. For lighter tampers (less than about 15 to 20 kips, many excavating and earth moving contractors have adequate equipment on hand to undertake this work. When the size of the tamper is in a range 20 to 40 kips, heavier lifting equipment is required, so much of this work is done by specialty contractors.

When the tamper exceeds 35 to 40 kips, specialized equipment will be necessary. This equipment could consist of a normal lifting crane modified with a large-diameter hoist for repeatedly raising and dropping the tamper. A large crane with a special hoist on the back for repeatedly raising and dropping a 64 kip tamper is shown in Figure 4-17. A specialized crane that has been developed for lifting tampers in the range of 64 to 100 kip is shown in Figure 4-18. The costs for deep dynamic compaction are much higher for these specialized pieces of equipment using the heavier tampers.

For planning purposes, the desired depth of improvement is selected, and then Equation 4-1 is used to predict the proper energy. After selecting a drop height, the mass of the tamper is then determined. The cost may be estimated using Table 4-4. Additional information on estimating costs are presented in FHWA GEC 1 (1995) and *GeoTechTools*.

In addition to the cost for deep dynamic compaction, there may be other factors:

- The construction of a working platform over soft ground
- Additional fill requirements to maintain the original grade
- Undercutting of weak deposits, such as zone 3 soils that don't respond to deep dynamic compaction
- Pre-construction trial test sections, and/or pre- and post-construction monitoring and testing

Though a granular blanket generally is not used on firm ground, on weak deposits, such as landfills, it may be necessary to import 1 to 3 feet of a crushed rock or granular material. This would add to the cost for deep dynamic compaction. If the ground subsidence is significant and the grade must be maintained, additional fill would be required to compensate for the induced settlement, which could be 5 to 10 percent of the thickness of the densified ground.

2.6 Case Histories

Two case histories are presented to illustrate typical applications.

2.6.1 Highway Embankment Constructed in Mine Spoil

2.6.1.1 Project Description

Interstate 65 in Jefferson County, Alabama, was extended over a mine spoil area for approximately 2,500 feet. At either end of the mine spoil area, the final grades required approximately 15 feet of new fill, but in the center, there was a cut of 45 feet. From this final grade, there would still be 50 to 120 feet of mine spoil below the roadway. The new roadway was to be a four-lane highway with a width of 182 feet.

2.6.1.2 Soil Conditions

The mine spoil is classified as a mixture of rock fragments of shale, siltstone, and sandstone embedded within the soil matrix of silt and sand, or clayey silt and sand. Approximately 50 percent of the mine spoil was larger than 2 inches.

The area was strip mined from 1977 to 1980. The initial subsurface investigations for the interstate occurred in 1983, and construction of the roadway started in 1984. Thus, the mine spoil was of relatively recent age at the time of construction.

Soil borings were made through the mine spoil using standard penetration tests (SPT). Excluding the very high SPT values where weathered chucks of rock were encountered, the majority of the mine spoil had SPT values ranging from 15-20 blows per 1 foot. There were some areas where the SPT values were as low as 5-10 blows per 1 foot and other areas as high as 25-35 blows per 1 foot. The variation in SPT value at depth is illustrated in Figure 4-24.

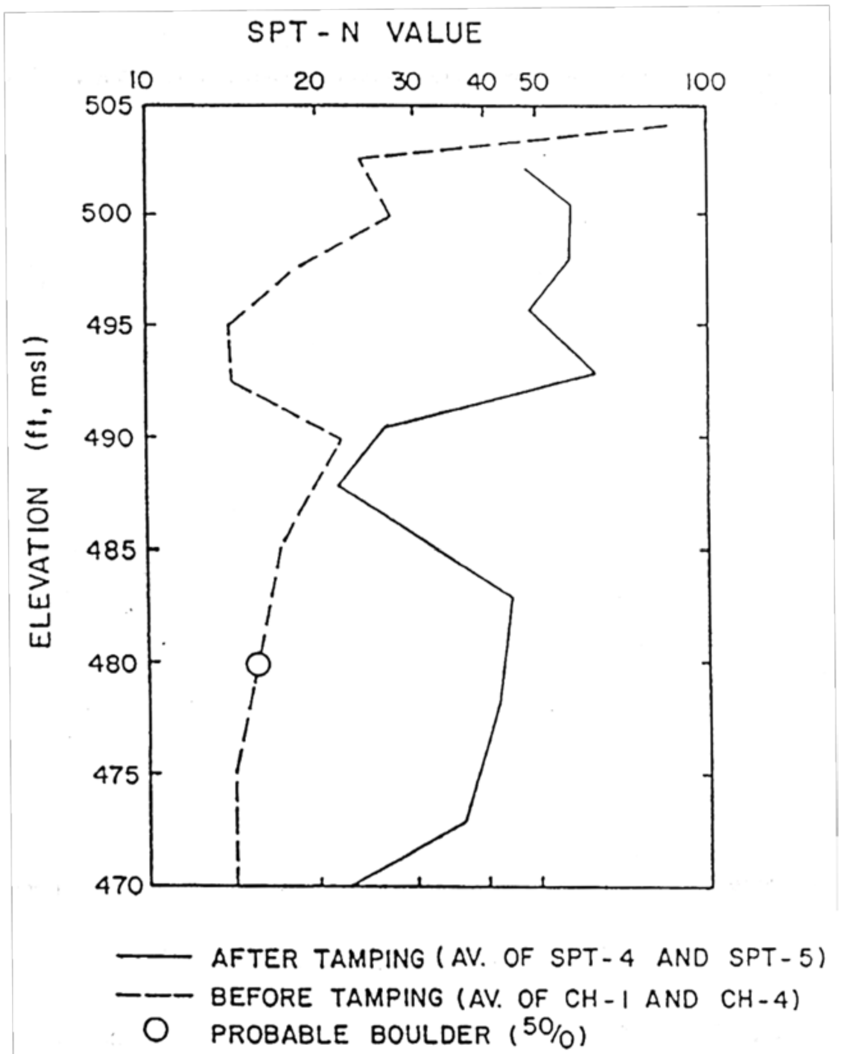


Figure 4-24. Increase in SPT values in a mine spoil after deep dynamic compaction.

Ground water was encountered at depths in excess of 100 feet below ground surface, and was not a factor on this project.

2.6.1.3 Design Concerns

The Alabama Highway Department and its consultants realized that there were large variations in material classifications, strengths, and compressibilities in relatively short vertical and horizontal distances over the entire site. For this reason, site improvement by dynamic compaction was recommended in order to achieve the following:

- Provide a more uniform subgrade for the new pavement
- Crush existing large boulders and rock fragments that are subject to deterioration
- Densify localized areas of loose/soft soil in the upper 25 to 30 feet, which could subside after highway construction

Excavation of 25 feet of mine spoil followed by recompaction with conventional compactors was considered an alternate to deep dynamic compaction. However, deep dynamic compaction was found to be significantly less expensive.

2.6.1.4 Predicted Densification Procedure

Using the guidelines presented in this chapter, the first step was to calculate the tamper mass and drop height for a desired depth of improvement of 25 to 30 feet (7.6 to 9 m).

Using D = 30 feet (9 m), n = 0.5 as average, and Equation 4-1,

- $9 \text{ m} = 0.5(WH)^{1/2}$
- $WH = 324$

From Figure 4-22, using W = 20 Mg (44 kip), H ranges from 15 to 31 m (49 to 103 feet); use H = 16.5 m (54 feet), since WH = 20 x 16.5 = 330 > 324 Mg-m.

The second step was to calculate the energy to apply. Using Table 4-3 as a guide, for Zone 2 type deposits

- E = 250 to 350 kJ/m^3 (5,200 to 7,200 ft-lbf/ft^2).

Because the deposits are already in a medium-dense condition, use E = 250 kJ/m^3 (5,200 ft-lb/ft^2). For a 9 m (30 feet) thick deposit, applied energy = 9 m *x* 250 kJ/m^3 = 2250 kJ/m^2.

The third step was to determine the grid spacing and number of drops. Assuming a 20 Mg (44 kip) tamper dropped 16.5 m (54 feet), a reasonable spacing would be 3 m (10 feet), as the tamper diameter is likely to be 2 m (6.5 feet). Using Equation 4-2, and assuming only one pass and an ironing pass, which will apply 250 kJ/m^2 energy, the number of drops can be computed.

$$(2250-250)\,kJ\,/\,m^2 \;=\; \frac{(20\,Mg)\,(10\,kN\,/\,Mg)\,(16.5\,m)\,(N)}{(3m)^2}$$

For the grid spacing of 3 m *x* 3 m, N = 5.5; or six drops at each grid point.

After dynamic compaction, the maximum SPT value according to Table 4-2 would be 25 to 35 for sandy silts, and 20 to 35 for clayey silts. Estimated induced settlement is 5 percent (9 m) = 0.45 m (1.5 feet).

2.6.1.4 Actual Densification Procedure

Based upon results of a test section, it was determined that densification to a depth of 7.6 to 9 m (25 to 30 feet) could be achieved using a 20-Mg (44-kip) tamper with a drop height of 19 m (62 feet). The high-level energy was applied using five drops at each grid point location, with a spacing of 3.0 m (10 feet) between grid points.

After the high-level energy was applied, the ground surface was leveled and an ironing pass completed using the same tamper with a drop height of 5.8 m (19 feet), a grid spacing of 1.8 m (6 feet), and one drop per grid point location.

This procedure resulted in an average applied unit energy of 2.1 MJ/m^2 (142 $ft\text{-}kip/ft^2$) for the primary energy application and an additional 0.36 MJ/m^2 (24 $ft\text{-}kip/ft^2$) during the ironing pass which is approximately the same energy previously calculated using Table 4-3.

2.6.1.5 Ground Improvement

In Figure 4-25, soil borings with SPT values made after dynamic compaction are compared to the borings with SPT values before dynamic compaction. This data indicates the improvements were obtained to depths of approximately 10.7 m (35 feet) and that the SPT values increased significantly.

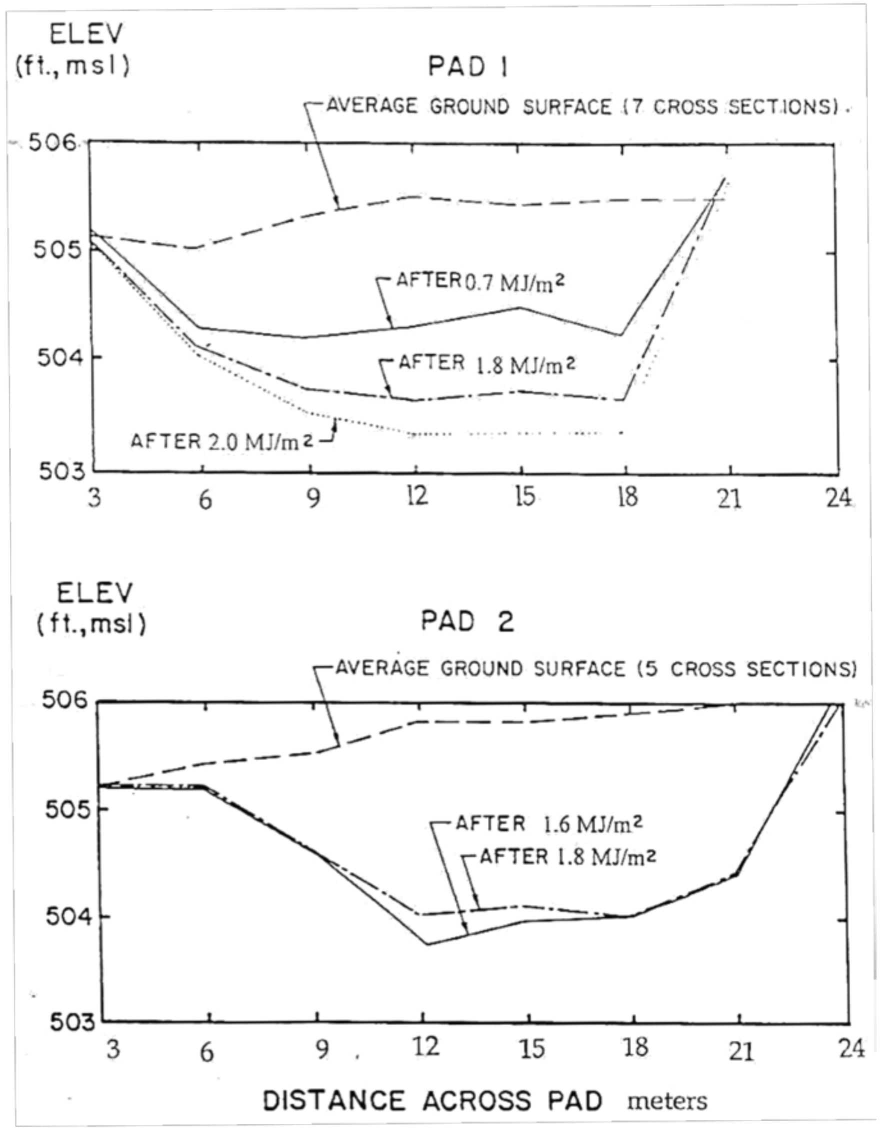


Figure 4-25. Induced settlement following deep dynamic compaction at the mine spoil project in Alabama.

Another indication of ground improvement was the amount of induced ground settlement by dynamic compaction. Within the test sections, ground elevations were taken on a grid pattern and measured following various levels of energy application. The data are summarized in Figure 4-26.

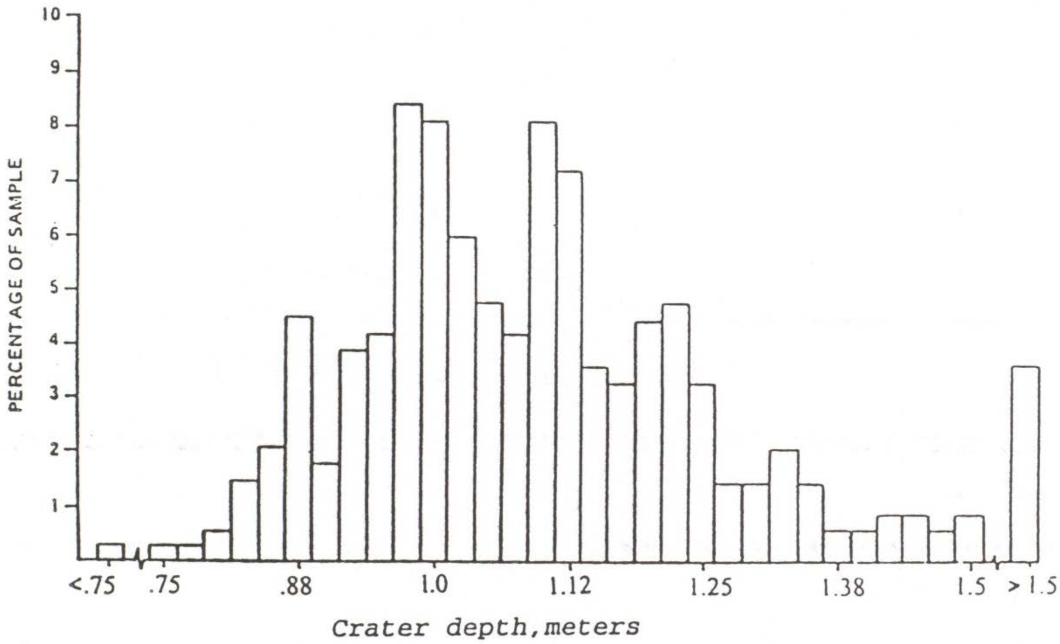


Figure 4-26. Statistical variation in crater depths at the mine spoil site in Alabama.

At test pad 1, the average ground subsidence following full application of the primary energy was approximately 0.6 m (2 feet). At test pad 2, approximately 0.6 m (2 feet) of induced settlement was observed after energy application, corresponding to about 90 percent of the prescribed energy.

During the production phase of dynamic compaction, the average induced ground settlement was 0.5 m (1.6 feet), although it could have been more because some fill was brought in during the leveling of the craters. In local areas, the average induced settlement was significantly higher. The variation in the crater depths that were observed in certain sections of the dynamically compacted area are illustrated in Figure 24. The normal average crater depth for the mine spoil was 1 to 1.1 m (3.3 to 3.6 feet), but crater depth as high as 1.5 to 2.7 m (5 to 9 feet) occurred in some locations, indicating a soft or void area. Additional high-energy tamping was undertaken in the soft area after ground leveling and placement of fill to raise the grade.

2.6.1.6 Contracting Procedure and Cost

A method specification was prepared for this project, and non-specialty, as well as specialty, contractors were allowed to bid. The project was awarded to an excavating contractor. After an initial 2-week trial period with some experimentation, the work proceeded on a reasonably good schedule. One hundred working days were required to dynamically compact approximately 37,200 m^2 (44,500 yd^2) for an average of 372 m^2 (445 yd^2) per day. When considering only production time, the average tamping rate was 63 impacts per hour for the primary phase and 67 impacts per hour for the ironing pass.

The project was bid on a price per drop that included an overall mobilization charge for all facets of the embankment construction. Therefore, the portion of the mobilization attributed to dynamic compaction is difficult to determine. The bid price per drop was $2.90 for the high-energy phase and $2.65 per drop for the ironing pass. Using the prices given and an estimate for the mobilization, the cost for the dynamic compaction was approximately $7.20 per m^2 ($6.00 per yd^2) of area treated. This amounted to about 4 percent of the total project cost.

The estimated cost for excavation of the upper 6.1 m (20 feet) of soil followed by placement in lifts and compaction would have been approximately 2.6 times the cost of the dynamic compaction.

2.6.2 Highway Embankment on Landfill Debris

2.6.2.1 Project Description

The Route 7 bypass around Manchester, Vermont, crosses two areas underlain by old refuse. The southern area, designated as area 1, is approximately 61 m (200 feet) long, and the planned embankment extended to a height of approximately 7 m (23 feet) above present grade. The northerly area, designated as area 2, is approximately 91 m (300 feet) long, and the new planned embankment extended to heights of 3 to 3.7 m (10 to 12 feet) above present grade.

2.6.2.2 Subsurface Conditions

Both landfills were covered with about 0.6 m (2 feet) of gravelly glacial till that was used as a cover material. Below this level, old landfill material was present to depths ranging from 1 to 3.4 m (3.3 to 11.2 feet), but averaging about 2 m (6.6 feet) in area 1. The landfill was described as consisting of miscellaneous materials, including metals, plastic, bags, glass, and trash. No paper, food, or other biodegradable materials were encountered within the landfill. Occasional seams or layers of silty sand were encountered within the trash, but these were probably thin layers of daily cover. Standard penetration tests ranged from 10 to 14 blows for 300 mm (1 foot), with some values as low as 7.

At area 2, the thickness of the trash was approximately 1 to 6 m (3.3 to 20 feet) and averaged 3 m (10 feet). The trash consisted of the same classification as area 1. The water table was determined to be at a depth of 5.5 m (18 feet) in landfill area 2.

Both landfills were underlain by a medium-dense to dense silty sand and gravel containing boulders and cobbles.

The age of the landfills at the time of dynamic compaction was determined to be approximately 14-18 years after closure. Ordinarily, this would mean the landfill was in the middle age of decomposition. However, the absence of organic materials within the trash indicated that it was in an older stage of decomposition. Because no methane gas was noted on the boring logs, it was likely that the decomposition of the highly organic materials was complete.

2.6.2.3 Design Concerns

When landfills decompose, a relatively loose structure is all that remains, creating the potential for significant total and differential settlement. For this reason, some method of ground improvement was necessary, and dynamic compaction was selected to reduce the potential for this predicted movement.

2.6.2.4 Predicted Densification Procedure

Using the guidelines in this chapter, the first step was to calculate the tamper mass and drop height for a desired depth of improvement ranging from 3.4 m (11.2 feet) maximum in area 1 and 6 m (20 feet) maximum in area 2.

For area 2:

Using Equation 4-1, with n = 0.4 for a landfill, and D = 6 m (20 feet),

- 6 m = 0.4 $(WH)^{1/2}$
- WH = 225 Mgm

From Figure 4-22, the desired energy can be obtained as the product of tampers ranging from about 13 to 16 Mg multiplied by drop heights ranging from 14 to 30 m.

For a 14-Mg tamper, use:

$$H = \frac{225\,Mg\,m}{14\,Mg} = 16\,m$$

For area 1:

Using Equation 4-1, with n = 0.4, D = 3.4 m, and W = 14 Mg,

$$H = \frac{(3.4/0.4)^2}{14\,Mg} = 5.2\,m$$

The second step was calculating how much energy to apply. Using Table 4-3 as a guide, E = 600-1100 kJ/m^3 for a landfill. Because the SPT values indicated a medium dense condition, E = 800 kJ/m^3 was selected.

Using the average fill thickness, the required average applied energy can be calculated:

- For area 2, E = (3 m) x (800 kJ/m^3) = 2400 kJ/m^2 = 2.4 MJ/m^2
- For area 1, E = (2 m) *x* (800 kJ/m^3) = 1600 kJ/m^2 = 1.6 MJ/m^2

The third step was determining the grid spacing and number of drops. Assume all the energy can be applied in one pass. For a 14-Mg tamper, the diameter is typically 1.6 m (5.2 feet), suggesting a grid spacing of 2.3 m (7.5 feet). Because the highway department planned on using a surface compactor following dynamic compaction, the ironing pass was eliminated. The number of drops can now be calculated using Equation 4-2.

For area 2, a grid spacing of 2.3 m, and 1 pass

$$2400kJ/m^2 = \frac{(14Mg)(10kN/Mg)(16m)(N)(1)}{(2.3m)^2}$$

where N = 5.66 drops or 6 drop per grid point.

For area 1, a grid spacing of 2.3 m, and 1 pass

$$1600kJ/m^2 = \frac{(14Mg)(10kN/Mg)(5.2m)(N)(1)}{(2.3)^2}$$

where N = 11.6 or 12 drops per grid point.

To reduce the number of drops, the drop height could be increased because the equipment provided for area 2 will have the capacity to lift the tamper to 16 m. To have the same number of drops (6) as for area 2, use this equation to calculate drop height:

$$1600kJ/m^2 = \frac{(14Mg)(10kN/Mg)(H)(6drops)(1)}{(2.3)^2}$$

where H = 10 m

After dynamic compaction, the maximum SPT value according to Table 4-2 would be anticipated to be 15 to 40.

The estimated induced settlement would be (20%)(3 m) = 0.6 m (1 feet) in area 2, and (20%)(2 m) = 0.4 m (1.3 feet) in area 1, based on experience with landfills.

2.6.2.5 Actual Densification Procedure

Before starting dynamic compaction, the site was leveled by lowering the elevation in the high portion of the site, then placing some of the debris in the lower portion of the site. Because the debris was variable, a 0.6-m (2 feet) blanket of silty sandy gravel was placed on the surface as a working mat.

The specifications required a 13.6-Mg (30 kip) (minimum) tamper, an 18-m (60 feet) drop in the shallow fill area, and a 27-m (90 feet) drop height in the deeper fill area. The contractor used a 14-Mg (31 kip) tamper with 18 and 27 m (60 and 90 feet) drop heights and elected to apply the energy in three phases. The first phase consisted of dynamic compaction on a grid basis with a spacing of 4.6 m (15.4 feet) between drop points. The second phase consisted of the same grid pattern, offset from the first by 2.3 m (7.5 feet) so as to be situated between the phase 1 points. The third phase consisted of energy applied at the phase 1 drop point locations. Seven drops were applied at each drop point location. This resulted in an average energy at the ground surface of 2.5 MJ/m^2 (169 $ft\text{-}kip/ft^2$) for area 1 and 3.75 MJ/m^2 (254 $ft\text{-}kip/ft^2$) for area 2, which is considerably more energy than required by the calculations, based on the presented guidelines. However, the specifications required a drop height much greater than that required by Equation 4-2, thereby resulting in more energy being applied.

Crater depths were monitored during dynamic compaction. In the first phase, the crater depths were typically 1 m (3.3 feet). In the third phase, which took place at the same location as the first phase, the crater depths were 0.5 m (1.6 feet). Heave measurements were taken adjacent to the drop point locations, and heave was not observed.

2.6.2.6 Ground Improvement

No soil borings were made after dynamic compaction. The initial plan was to install settlement plates along the completed sections of the roadway, but this was not undertaken. Discussions with the highway engineer indicate that the pavement sections have performed well in this area and there is no evidence of settlement.

2.6.2.7 Contracting Procedure and Cost

A method specification was prepared by the agency for this project. The specification included the tamper weight, drop heights ranging from 18 m (59 feet) in the shallow fill areas to 27 m (88.6 feet) in the deep fill areas, plus the number of phases of energy application and the spacing between the drop point locations. The number of drops at each location was specified to range from a minimum of 6 to a maximum of 10.

A specialty contractor was awarded this project and was able to demonstrate that 7 drops per phase were sufficient to achieve satisfactory densification. The cost for dynamic compaction was \$10.25 per m^2 (\$8.57 per yd^2). This cost does not include the placement of the 0.6 m (2 feet) gravel blanket used as a working mat.

3.0 VIBRO-COMPACTION

3.1 Feasibility Considerations

Vibro-compaction can be used to achieve a number of design objectives, as discussed in Chapter 1. This section discusses applications for transportation facilities, as well as advantages, disadvantages, limitations of the system, and feasibility.

3.1.1 Applications

This section focuses on the use of vibro-compaction as a solution to problems related to transportation projects. Thousands of vibro-compaction projects have been completed in the United States, with about 10 percent being transportation related.

For transportation projects, vibro-compaction can be used to treat problems related to the following:

- Foundation soils beneath proposed structures
- Highway embankment fills
- Tunnels - compaction of overburden soils
- Densification of artificial tunnel islands
- Mitigation of liquefaction potential for transportation applications:
    - Compaction to stabilize pile foundations driven through loose granular materials
    - Densification for abutments, piers, and approach embankment foundations
- Compaction of underwater embankment fills
- Compaction in areas of potential cavities beneath embankments to pre-settle and fill such voids prior to construction of a structure

3.1.2 Advantages and Disadvantages

3.1.2.1 Advantages

As an alternative to deep foundations, vibro-compaction is usually more economical and often results in significant time savings. Loads can be spread from the footing elevation, thus minimizing problems from lower, weak layers. Densifying the soils with vibro-compaction can considerably reduce the risk of seismically induced liquefaction. Vibro-compaction is a cost-effective alternative to removal and replacement of poor load-bearing soils. The use of vibro-compaction allows maximum improvement of granular soils to a depth of about 165 feet, with generally recommended depth of about 100 feet. The vibro-compaction method is effective both above and below the natural groundwater level.

3.1.2.2 Disadvantages

The primary disadvantage of vibro-compaction is that it is effective only in granular, cohesionless soils. The realignment of the sand grains and, therefore, proper densification generally cannot be achieved when the granular soil contains more than 12 to 15 percent silt OR more than 2 percent clay. The maximum depth of 165 feet may be considered a disadvantage, but there are very few construction projects that will require densification to a greater depth.

Like all ground modification techniques, a thorough soil investigation program is required. A more detailed soils analysis may be required for vibro-compaction than for a deep foundation project. This is because the vibro-compaction process utilizes the native soil to the full depth of treatment to achieve the end result. A comprehensive understanding of the total soil profile is therefore necessary. A vibro-compaction investigation will require continuous standard penetration tests (SPT), and/or cone penetrometer (CPT), as well as gradation tests to verify that the soils are suitable for vibro-compaction.

3.1.3 Feasibility Evaluations

3.1.3.1 Geotechnical (In Situ Soil Gradations)

Vibratory compaction of soils is most effective on granular materials having little to no fines or low cohesion or plasticity. A quick assessment of the suitability of granular soils for treatment by vibro-compaction was proposed by Degen (1997) on the basis of the Unified Soil Classification System (USCS) and is shown in Table 4-5.

Table 4-5. Suitability Assessment of Granular Soils for Vibro-compaction

Soil Type	USCS	Comments on Suitability for VC
Gravel, well graded	GW	Well suited for VC, potential penetration difficulties with less powerful machines
Gravel, poorly graded	GP	If $D_{60}/D_{10} \leq 2$ compaction only marginal (trail compaction recommended)
Gravel, silty or clayey	GM, GC	Compaction not possible if clay content >2% and silt content >10%
Sand, well graded	SW	Ideally suited
Sand, poorly graded	SP	If $D_{60}/D_{10} \leq 2$ compaction only marginal (trail compaction recommended)
Sand, silty	SM	Compaction inhibited if silt content >8%
Sand, clayey	SC	Compaction inhibited if clay content >2%

Sources: Kirsch and Kirsch 2010 and Degen 1997

The suitability of a soil for vibro-compaction methods has generally been determined on the basis of grain size distribution, as shown in Figure 4-27.

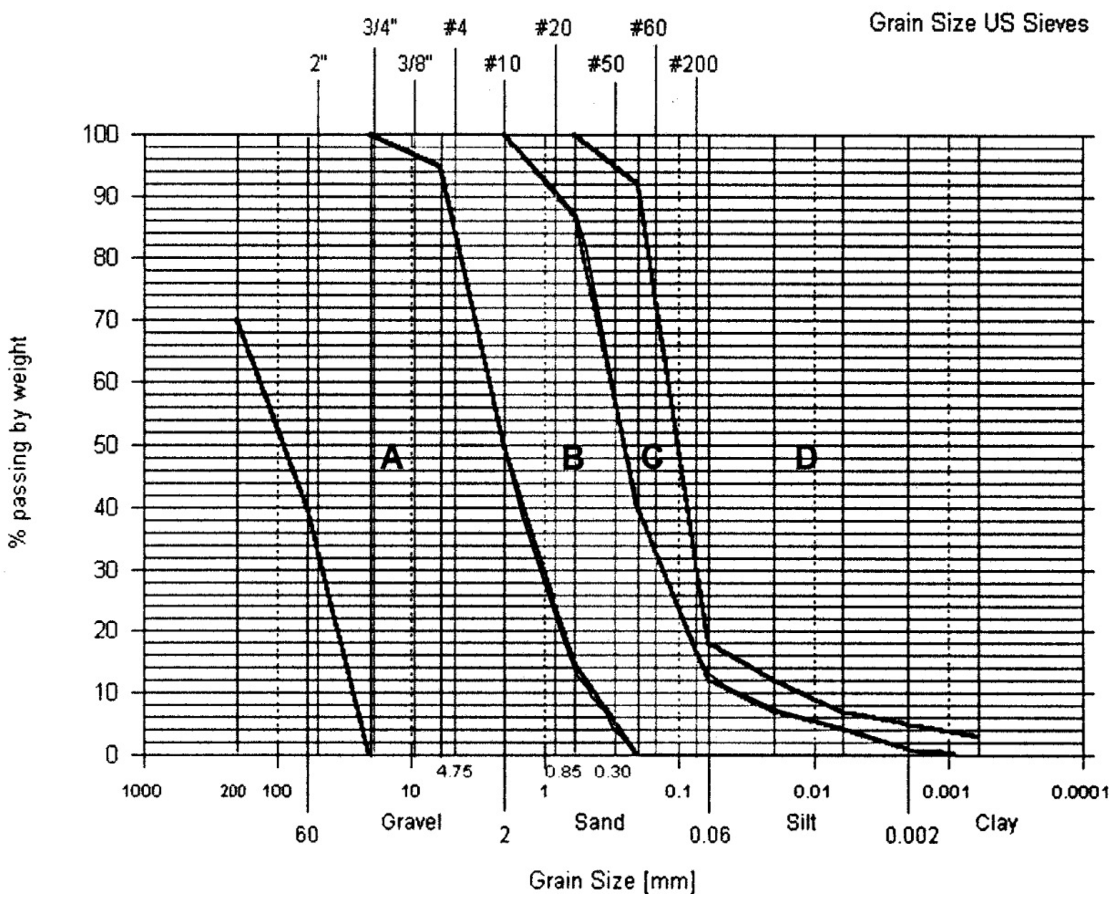
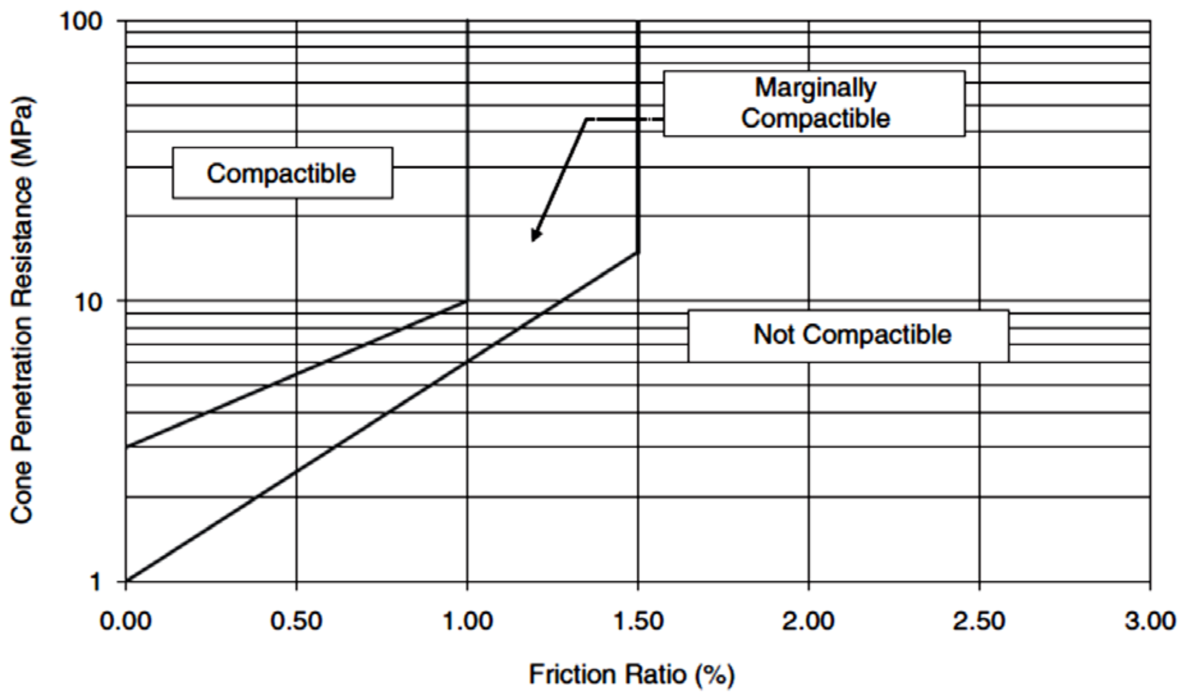


Figure 4-27. Range of soil types treated by vibro-compaction.

Soils with grain size distribution curves lying entirely within zone B are ideally suited for vibro-compaction with fines content below 10 percent. Soils in zone A are well compactible but the increasing gravel content are resultant high permeability may affect the ability of the depth vibrator to penetrate to desired depths. If the grain-size distribution curve falls in zone C, it is advisable to backfill with gravel in lieu of sand during the compaction process. This will improve the contact between the vibrator and the treated soil and drastically increase compaction time. The soils with grain size distribution curves partly or entirely in zone D are not readily compactable by vibro-compaction. However, these soils can be improved by vibro-replacement, as described in Chapter 5 Aggregate Columns.

An alternative way to assess suitability for vibro-compaction is the use of cone penetration tests. An advantage of using cone penetration test is that continuous readings of cone tip resistance and sleeve friction can be obtained as compared to spot samples collected for grain-size analyses. The suitability classification proposed by Massarsch (1991) is shown in Figure 4-28.



After Massarsch 1991

Figure 4-28. Soil compactibility based on cone penetration resistance and friction ratio.

For cohesionless soils with natural dry densities less than their maximum dry densities, the influence of vibrations will result in a rearrangement of their grain structures. Under the influence of induced vibrations, the inter-granular forces between the grains in non-cohesive soils are temporarily nullified. The grains are then rearranged, unconstrained, and unstressed under the action of gravity to a more dense state. The void ratio and compressibility of the soil treated by vibratory means will be decreased, and the angle of shearing resistance increased. The treated, compacted soil is capable of sustaining higher bearing pressures for the same settlements as the untreated soil, and undergoes smaller settlement for the same bearing pressure, with the settlement generally being only elastic. The achievable reduction in void ratio depends on grain shape, soil composition, and vibration intensity (Moseley and Priebe 1993).

By advancing the vibrator to the desired level and withdrawing it from the ground in a specific manner, the granular soils are compacted by the horizontal vibration forces. A compact soil cylinder is thus formed, with the diameter determined by the grain size distribution, the soil density, and the vibrator characteristics. By arranging compaction points in suitable patterns, soil masses can be compacted homogeneously.

The increased density of the granular soils results in the downward movement of the soil around the vibrator and creates a cone-shaped depression at the surface. This depression must be continuously in-filled with granular fill material. If on-site material is used, then the original ground surface will be lowered. Alternatively, ground level can be maintained by adding imported granular fill material, which is compacted simultaneously with the natural soil.

The vibro-compaction process subjects the soil mass to high accelerations during compaction. These levels of dynamic strain are unlikely to be repeated, even under earthquake loading. Provided that the design earthquake criteria are not exceeded during a seismic event, the treated ground can be expected to perform as designed.

Soil compaction, as achieved in the vibro-compaction process through the rearrangement of soil particles, is not possible in cohesive, fine-grained soils. The cohesion between the particles prevents rearrangement and compaction from occurring.

3.1.3.2 Environmental Considerations

The dry method of vibro-compaction is only viable in clean, sandy, fully saturated soils. The great majority of vibro-compaction projects are therefore accomplished by the wet method. Although the vibro-compaction technique is used for densifying primarily granular soils, the jetting water effluent will nevertheless require temporary construction provisions to contain and dispose of any silt and clay in the effluent. With current awareness of potential environmental problems, geotechnical exploration programs should include not only the classification of the soil type and location of groundwater, but also the examination and classification of any potential contaminants in the soil and groundwater. If contaminants are uncovered in the original exploration program, a determination should be made as to whether they can be treated at the site during the vibro-compaction program. If this is not the case, then an alternative densification program, such as the dry bottom feed stone column technique (which does not produce jetting water effluent), or other solutions, should be considered.

3.2 Construction and Materials

The vibro-compaction process uses crane-mounted depth vibrators and appropriate backfill material. This section discusses construction equipment and the suitability of backfill material.

3.2.1 Construction

The equipment used to achieve the necessary densification are high-powered, probe-type vibrators ranging from 12 to 16 inches in diameter and 10 to 15 feet in length, as shown in Figure 4-29 and Table 4-6.

Courtesy of Hayward Baker

Figure 4-29. High-powered, probe-type vibrator utilized in vibro-compaction.

Table 4-6. Specifications of Several Vibrators

Vibrator	Length m	Dia. mm	Weight kg	Motor kW	Speed rpm	Ampl. mm	Dynamic Force kN
Bauer TR13	3.1	300	1000	105	3250	6	150
Bauer TR85	4.2	420	2090	210	1800	22	330
Keller M	3.3	290	1600	50	3000	7.2	150
Keller S	3.0	400	2450	120	1800	18	280
Keller A	4.3	290	1900	50	2000	13.8	160
Keller L	3.1	320	1815	100	3600	5.3	201
Vibro V23	3.6	350	2200	130	1800	23	300
Vibro V32	3.6	350	2200	130	1800	32	450

Source: Layne Christiansen Company
Note: See table at front of manual for SI conversions.

A set of rotating eccentric weights housed inside the probe is mounted on a vertical shaft. Vibrations (induced by rotating these weights) are produced close to the bottom of the unit. A motor located within the casing, as shown in Figure 4-30, drives the rotating shaft.

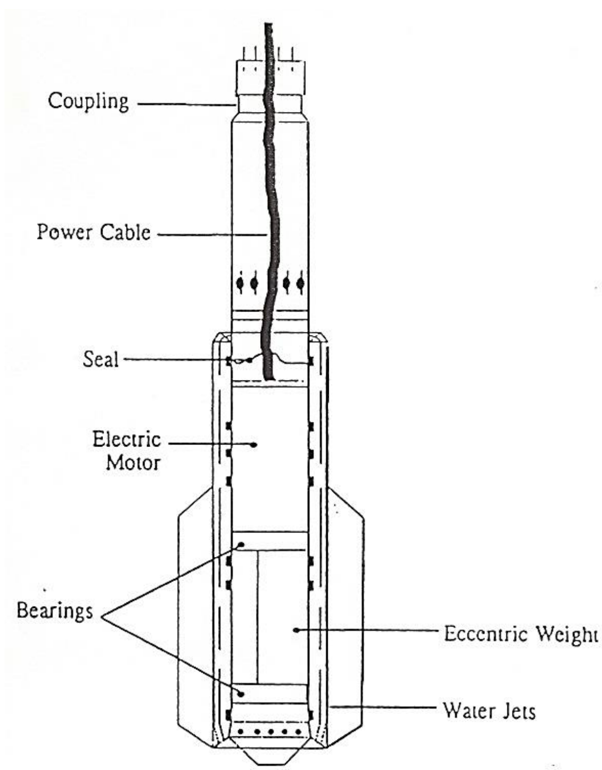


Figure 4-30. Cross-section of a typical vibrator.

To drive the assembly shown in Figure 4-30, an electrically driven motor is usually employed, driven by motors typically in the 100-130 kW range. The vibrations produced by these units are generated at the nose of the unit and, as a result of the rotation of the weights, emanate radially in the horizontal plane away from the unit. The units now in general use generate dynamic forces from 33,750 to 100,000 lbs (150 to 450 kN) at frequencies ranging from 1,800-3,200 rpm. However, for vibro-compaction, vibrators operating at lower frequencies will *usually* produce better densification results than those operating at higher frequencies. This is because low frequency vibrators usually have a higher amplitude, which translates into a greater compactive effort. Additionally, the natural frequency of most densifiable soils is closer to 1500 rpm than to 3000 rpm. Selected available vibrators and some of their operating characteristics are described in Table 4-6.

Follower tubes of a similar or lesser diameter are attached to the vibrating unit in order to extend its length to allow treatment of soils at depth. The follower tubes are attached to the vibratory unit by means of an isolation coupling, thus preventing the vibrations from traveling up the follower tubes, negating the problem of energy losses at depth.

The complete assembly is supported from a standard crane (Figure 4-31), a specially built hydraulic crawler crane, or a crane that is mounted on a barge (Figure 4-32), depending upon the site conditions.

Figure 4-31. Vibrator suspended from a conventional crane.

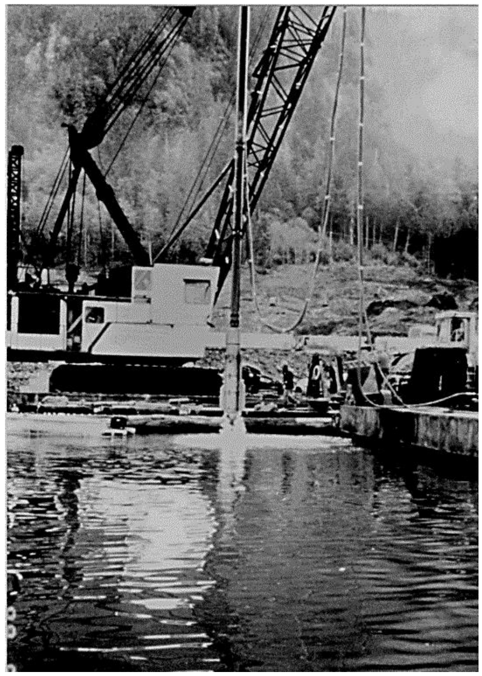


Figure 4-32. Vibrator suspended from a barge-mounted crane.

The vibro-compaction operation necessitates the use of water- or air-jetting to facilitate the penetration of the vibrator and to densify the soil. Therefore, water- or air-feed hoses, as well as water or air pumps, are also required.

3.2.2 Materials

In order to transmit vibrations from the vibrator into the in situ soil and achieve adequate compaction, it is necessary to supply sufficient backfill material to fill the void created by the densification process. Fine sands, coarse sands, rounded gravel, crushed stone, recycled aggregate, and slag have all been used as backfill material. Slag has the advantage of being inexpensive in some locations, but does not settle as fast as other material with comparable gradation. Coarse materials with little or no fines make the best backfill. However, if the particle size becomes too large, the gravel will arch in the annular space between the follower tube and the void, preventing backfill from reaching the vibrating tip.

The suitability of the backfill appears to be a function of the backfill quantity that can accumulate around the vibrating tip in a fixed period of time. The backfill gradation is the most significant factor controlling the rate at which the backfill settles through the wash water and accumulates around the tip. A rating system has been developed to judge the suitability of backfill material for vibro-compaction, based on the settling rate of the backfill in water and project experience (Brown 1977). This rating is dependent on a “suitability number” and is a function of the grain size diameters of the backfill material. The equation used in this calculation is as follows:

$$Suitability\ No. = 1.7\sqrt{\frac{3}{(D_{50})^2} + \frac{1}{(D_{20})^2} + \frac{1}{(D_{10})^2}} \quad \text{[Eq. 4-3]}$$

where D_{50}, D_{20}, and D_{10} are the grain size diameters, in millimeters, at 50 percent, 20 percent, and 10 percent passing. The qualitative categories of backfill using utilizing this rating system are listed in Table 4-7.

Table 4-7. Backfill Evaluation Criteria

Suitability Number	**Rating**
0 to 10	Excellent
10 to 20	Good
20 to 30	Fair
30 to 40	Poor
>50	Unsuitable

Source: Brown 1977

The quality of backfill material affects the allowable withdrawal rate of the vibrator. Within reasonable limits, the lower the suitability number, the faster the backfill will settle, and the faster the vibrator can be withdrawn and still achieve acceptable compaction. Backfill normally consists of material graded as sand, or sand and gravel, with less than 10 percent by weight passing the #200 sieve, and containing no clay.

3.3 Design

Similar to other ground modification methods, design of a vibro-compaction program requires definition of the problem, identification of all possible solutions, and the development of performance requirements for the improved soil. Depending on the type of project being designed, the prime consideration could be total or differential settlement, bearing capacity, or a seismic/liquefaction resistance requirement.

3.3.1 Design Considerations

If loose granular soils are identified as the site problem, then densification by vibro-compaction will be a potential technical solution, especially if the loose deposit is deeper than 35 feet as measured from the surface. The relationship between penetration resistance from subsurface investigations and soil properties that are useful in assessing the current density and the feasibility of improvement, as well as in identifying the potential targets and performance requirements for the improved soil, are indicated in Table 4-8.

Table 4-8. Penetration Resistance and Sand Properties

Penetration Resistance	Very Loose	Loose	Medium Dense	Dense	Very Dense
SPT N-value (blows/foot)*	< 4	4 to 10	10 to 30	30 to 50	> 50
CPT cone resistance (kg/cm^2 or tsf)	< 50	50 to 100	100 to 150	150 to 200	> 200
Equivalent Relative Density (%)*	< 15	15 to 35	35 to 65	65 to 85	85 to 100
Dry Unit Weight (pcf)	< 90	90 to 100	100 to 115	115 to 130	> 130
Friction Angle, degrees	< 30	30 to 3235	35 to 40	40 to 45	> 45
Cyclic Stress Ratio Causing Liquefaction**	< 0.04	> 0.04 to 0.12	> 0.12 to 0.33	> 0.33 to 0.40	
Shear Wave Velocity (ft/s)***	< 400	400 to 525	525 to 650	650 to 740	> 750

* Normally consolidated sand
** Seed et al. 1983
*** Debats and Sims 1997

If vibro-compaction is selected as the improvement method, the following parameters must be determined:

- Gradation of the in situ soils, including silt and clay content
- Existing relative density, or looseness, of the in situ soils
- Required density improvement necessary to solve the project's requirements and, once determined, whether this improvement is feasible

3.3.2 Design Procedure

The significant engineering properties of a granular soil – compressibility, shear resistance, permeability, resistance to dynamic loading – are largely dependent on the state of compaction, typically expressed in terms of relative density for clean granular materials. The term “relative density,” or D_r, is defined as follows:

$$D_r = \left[\frac{\gamma_d - \gamma_{d(\min)}}{\gamma_{d(\max)} - \gamma_{d(\min)}}\right] x \frac{\gamma_{d(\max)}}{\gamma_d} x100\% \qquad \text{[Eq. 4-4]}$$

where,

γ_d = dry unit weight of the soil in its natural state

$\gamma_{d(\min)}$ = dry unit weight of the soil in its loosest state

$\gamma_{d(max)}$ = dry unit weight of the soil in its densest state

In this calculation, $\gamma_{d(min)}$ and $\gamma_{d(max)}$ should be determined in accordance with current ASTM procedures [ASTM D-2049].

High relative density corresponds to high bearing resistance with low settlement. For seismic loading, resistance to liquefaction in granular soil is a function of relative density. In earth-retaining problems, active pressure decreases and passive resistance increases, as relative density increases. With vibro-compaction, the angle of internal friction is increased on average between 5 and 10 degrees, resulting in much higher shear resistance. The stiffness of the improved soils is increased, and consequently settlements are greatly reduced.

In addition to soil gradation, the area influenced (the tributary area) by each compaction point for a specified relative density depends on the compaction method used and the specific characteristics of the vibrator, which may not be known in advance. As shown in Table 4-6, vibrator characteristics vary widely.

The approximate relationships between relative density, soil type, and treatment area for a specific vibrator are shown in Figure 4-33. It is unlikely, even for heavy loading, that it will be necessary to achieve a relative density above 85 percent.

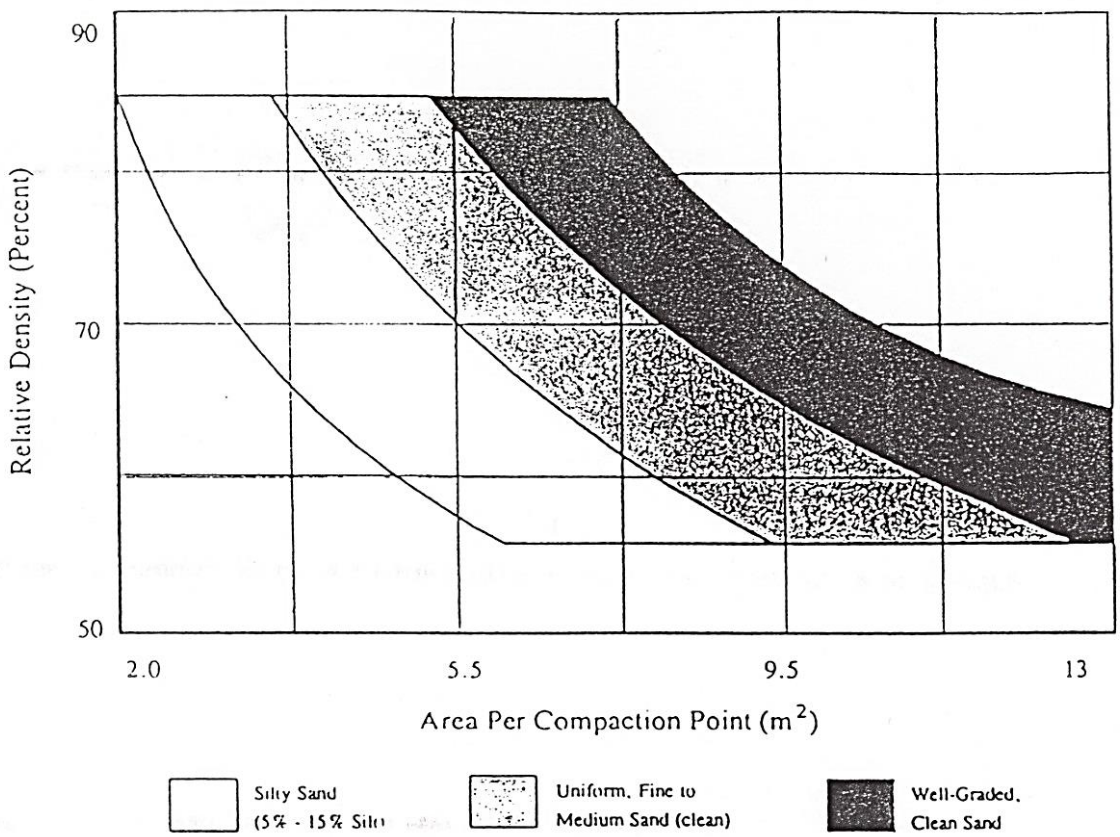


Figure 4-33. Approximate variation of relative density with tributary area.

A chart useful in estimating the probable level of improvement that can be obtained by vibro-compaction is shown in Figure 4-34. It is based on the lower bound soil gradation (silty sand) indicated in Figure 4-33. Similar charts can be developed for coarser granular soils from Figure 4-33.

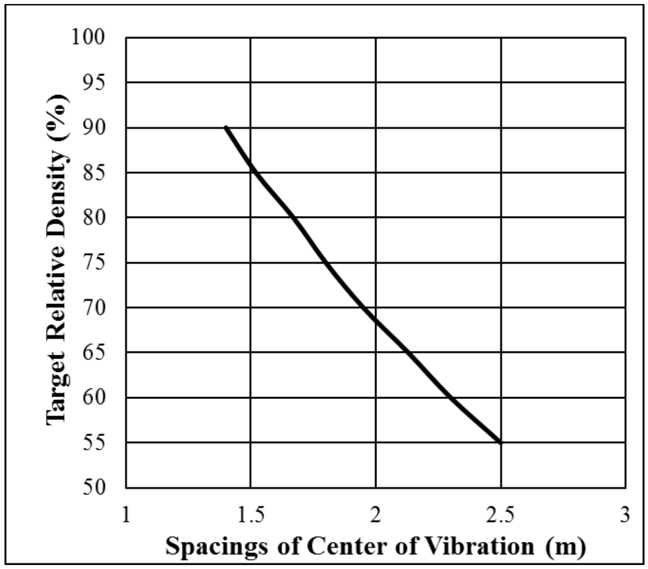


Figure 4-34. Relative density versus probe spacing for silty sands.

The project designer is responsible for setting the requirements for the project with an appropriate safety factor and the best method of confirmation testing. For most vibro-compaction projects, the following performance criteria should be considered:

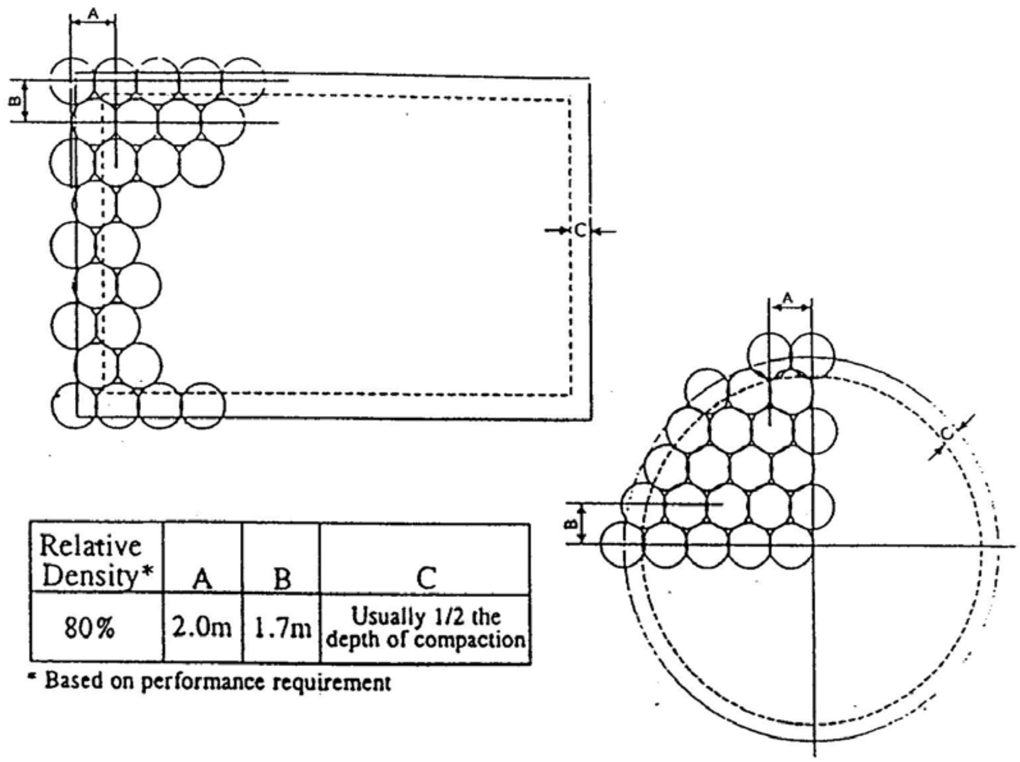
- 60 percent relative density for floor slabs, flat bottom tanks, embankments
- 70 to 75 percent relative density for column footings, bridge footings
- 80 percent relative density for machinery and mat foundations

3.3.2.1 Probe Spacing and Patterns

A typical vibro-compaction program is designed with various probe spacing and patterns. The distance between compaction points is critical, as the density generally decreases as the distance from the probe increases. Stronger vibroprobes allow for wider spacing under the same soil conditions.

The area compaction point pattern also affects the densification. An equilateral triangular pattern is primarily used to compact large areas, since it is the most efficient pattern. The use of a square pattern instead of an equilateral triangular pattern requires 5 to 8 percent more points to achieve the same minimum densities in large areas.

Given the in situ soil gradation and relative density required, the spacing of compaction points can be determined. Typical area patterns and spacing for 80 percent relative density requirements are illustrated in Figures 4-35 and 4-36. The spacing of the vibro-compaction points would be wider for a lower relative density requirement.



Relative Density*	A	B	C
80%	2.0m	1.7m	Usually 1/2 the depth of compaction

* Based on performance requirement

Figure 4-35. Typical compaction point spacing for area layouts.

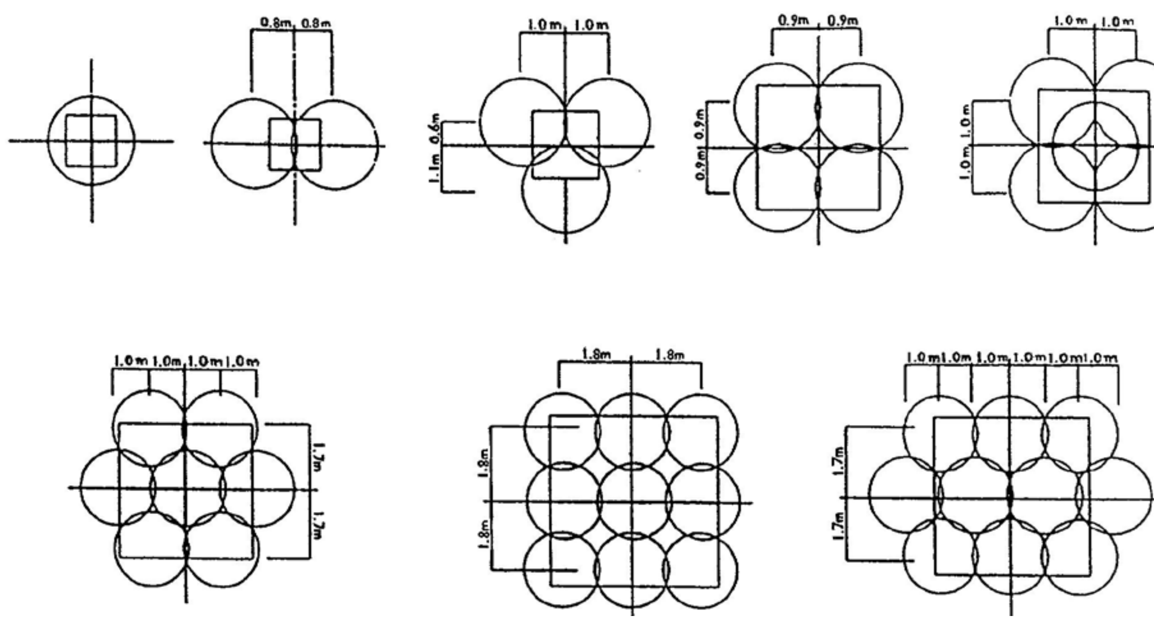


Figure 4-36. Typical compaction point layouts for column footings.

3.3.2.2 Performance Requirements

Performance requirements, such as total or differential settlement, bearing resistance or reduced liquefaction potential, can all be related to a desired in situ relative density. After compaction is completed, the effectiveness of compaction is normally evaluated to verify contractual compliance and to verify that the compacted soils will perform adequately under the intended loads or seismic event. A number of testing methods have been used, including Standard Penetration Testing, cone penetrometer soundings, shear wave velocity, or load tests. These methods, and their advantages and disadvantages, are described in Chapter 5. The potential for liquefaction can be evaluated using SPT blow counts and the cyclic shear stress ratio (CSR) at any depth.

3.4 Construction Specifications and Quality Assurance

3.4.1 Contracting Procedures

Vibro-compaction may be performed under either a method-type specification or a performance-type specification. Under a method-type specification, the specifying agency details a specific procedure and pattern spacing to achieve the required improvement. Bids are invited from contractors suitably equipped to perform the work. With this type of specification, the specifying agency assumes the risk, and a full knowledge of the ground improvement technology and equipment is required. If this knowledge is not available within the specifying agency, a method type specification is not advised.

Under a performance-type specification, the required end result is specified and the contractor assumes responsibility for achieving it. This approach does not require in-depth knowledge within the specifying agency. The contractor has the flexibility of selecting the procedure and pattern spacing to meet the design criteria.

Specifications and contracting procedures for vibro-compaction have changed significantly over the years. Where once the specifications stated a specific procedure and pattern spacing, variances in equipment and methods today favor placing the responsibility for achieving the required improvement on the contractor. Whereas the vibro-compaction method itself may still be specified, the contractor adopts the procedure and pattern spacing to achieve project objectives. Most vibro-compaction specifications today are performance based.

This section discusses contracting methods and quality control and inspection procedures. A guide to the preparation of a typical specification is included. Since the responsibility for achieving the design criteria for the ground improvement usually rests with the contractor under a performance specification, the focus of this chapter reflects this norm.

Most vibro-compaction projects require a certain degree of densification, which can be specified as follows:

- Minimum or average percent relative density

- Minimum or average (spt) blow count
- Minimum or average cone penetration resistance
- Minimum or average size of gravel or sand column
- Minimum amount of backfill material added
- Minimum load bearing requirement

All of the above have been used in past projects and, for the most part, have been successful. However, the best specification is one that allows for some variance of results within specified limits. Also, past experience has shown Standard Penetration Test blow counts to be misleading in certain stratifications and that specific percentage degrees of density are difficult to measure.

The technical literature has shown evidence that verification testing procedures can give misleadingly low results if performed immediately after densification, and that results can increase significantly with time (Mitchell and Solymar 1984; Debats and Sims 1997). A minimum wait of 5 days is recommended before performing verification testing, but a wait of about 10 days is preferable. The effectiveness of soil improvement with time after treatment should be considered in performing tests and interpreting test results.

A guide performance specification can be found in *GeoTechTools*. The format of the guide specification is deliberately generic. The responsible party should be inserted as appropriate when developing specifications for a particular project. Italicized terms or descriptions allow for flexibility to adapt to the specific requirements of the project to be improved by vibro-compaction. Where necessary, additional explanatory notes are included.

3.4.2 Quality Assurance and Monitoring

The quality assurance plan and inspection activities are developed well in advance of the vibro-compaction work. The duties of the Contractor and the Owner/Engineer with respect to QA/QC are dependent on the type of specification under which the work is being accomplished.

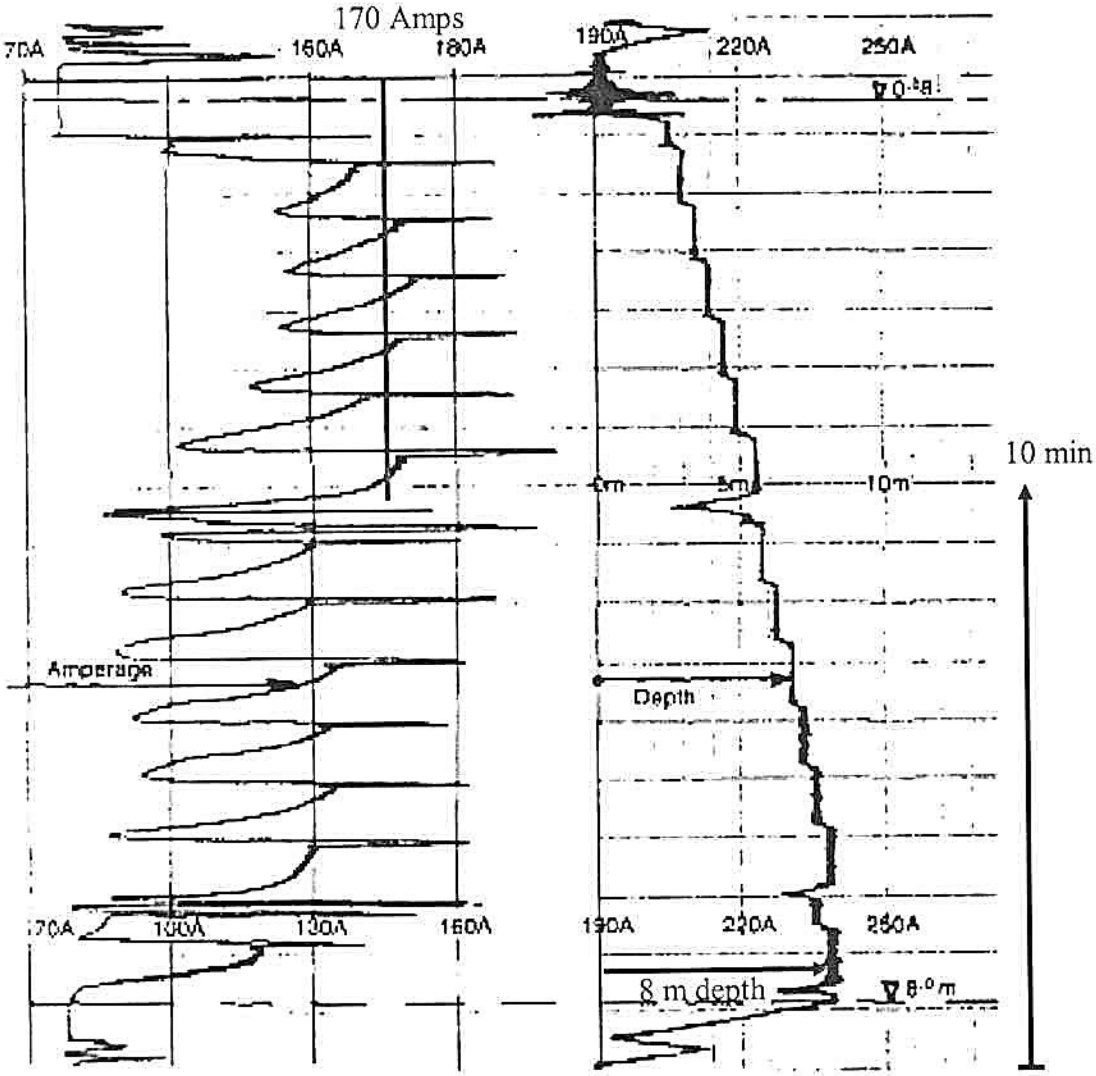
Under a method specification, development of the QA/QC plan, review of plans and specifications, and acceptance of backfill material are the responsibility of the Owner's Engineer. During performance of the work, the Owner's Engineer is responsible for all on-site inspection and testing. (Under a performance specification, these latter responsibilities lie with the Contractor). The Contractor provides the Owner's Engineer with testing results to verify that improvement criteria are being met. Under a performance specification, the

Owner is generally obligated to perform some independent verification testing based on the results supplied by the Contractor.

Under either type of specification, the inspection process during the actual compaction process should include the following:

1. Verification that probes penetration depth is acceptable
2. Verification that probe withdrawal rate is acceptable
3. Monitoring the probe penetration rate to obtain a rough indication of the type and density of soil penetrated
4. Verification that compaction points are at the proper locations
5. Monitoring the volume of backfill added to obtain an indication of the densities achieved
6. Verification that backfill gradation is acceptable
7. Monitoring of ammeter or hydraulic pressure readings to verify that the build-up is sufficient
8. Verification that the probes are operating at appropriate speeds
9. Verification that induced vibrations are not excessive when operating close to existing structures

Examination of the data logger records are a valuable QC tool and can be used to monitor uniformity of the compactive effort with depth. The length of time spent at each stage of compaction (1.5 to 2.0 feet) depends on the soil reaction and is shown on the right side of Figure 4-37, which measures from the bottom up. Generally the finer the soil, the longer the time required to achieve the same degree of compaction. The electrical current drawn by the motor increases as the soil around the vibroprobe densifies, as shown on the left side of Figure 4-37.



Debats and Sims 1997

Figure 4-37. Typical data logger results: amps versus depth (left) and time versus depth (right).

During the compaction process, the adequacy of compaction is periodically verified for quality control and acceptance purposes. These checks verify contractual compliance and compacted-soils performance under the intended loadings. A number of methods are used, including borings with SPTs, static CPTs, measurement of the surface subsidence, density measurements on undisturbed samples, and downhole nuclear densimeters. Each method has certain advantages and disadvantages.

The SPT is the most widely available method and the most widely used. However, it is also the least reliable method for estimating potential settlements, bearing capacity, and relative density of the compacted soils. SPT resistance N values are variable depending upon a number of factors. There is also significant scatter in the correlations of SPT resistance with relative density and with the soil properties needed to estimate settlement and bearing capacity. In addition, if data are obtained before pore pressures have dissipated, the penetration resistance will not be representative of the actual degree of soil improvement. SPT data are usually taken at 5 foot intervals, which is inadequate to properly evaluate the vertical variability of the vibro-compaction. However, this can be overcome by specifying continuous sampling, as is frequently the case.

The static CPT overcomes most of the disadvantages encountered with the SPT, and is considered the best available QA/QC method. The CPT is particularly advantageous since it is relatively inexpensive and can be used directly to estimate settlements in compacted areas. The cone resistance, however, will underestimate the degree of improvement if excess pore pressures are present.

Measurement of surface subsidence is an excellent way of monitoring the average increase in relative density, when the fill material is obtained from the compacted area. This method can also be used to check compaction of large areas if the quantity of imported fill is known. As a practical matter, it is difficult to accurately verify compaction achieved for footings with this method, and it is not possible to check for the minimum compaction achieved.

Downhole nuclear densimeters offer an alternate method for verifying final densities, but have not been used enough to establish their advantages and disadvantages relative to vibro-compaction. With this method, a small diameter aluminum pipe is placed in the ground to the planned compaction depth prior to compaction. Before and after compaction, a site-calibrated nuclear probe is lowered down the casing to obtain a continuous density-moisture- content profile. This method indicates the density within approximately 150 mm (6 in.) of the aluminum pipe.

3.5 Cost Information

3.5.1 Cost Components

Using the criteria described in Section 3.3, vibro-compaction point spacing can be determined. The total area requiring improvement can then be divided by the effective area of each point to determine the number of vibro-compaction locations required. In estimating costs, it is important to include the perimeter zone outside the limits of loaded area or influenced by vibro-compaction in the surface area calculation so that the project requirements are accurately matched. The depth of improvement required can then be multiplied by the number of points to determine budget footage of vibro-compaction. It is normally more economical to lower the entire site elevation by the vibratory compaction effort rather than add granular backfill from the surface. A typical price per linear foot of vibro-compaction would be $5 when no backfill is placed around the probe and $8 when granular backfill is added. The specific backfill cost will vary significantly on a local basis. In addition, mobilization/demobilization costs should be added. Other costs that should be considered include the following:

- *Surface densification.* With the lack of overburden restraint, the upper 3 feet of soil will have to be densified by conventional surface compaction methods.
- *Additional fill to raise the site to the required grade and, in the case of no added backfill, to compensate for the site's depression.* This cost will depend on the looseness of the in situ soil and the specified degree of densification.
- *Verification testing.* Standard Penetration Tests (SPT) are normally specified and, to ensure uniformity, some tests should be continuous. On large projects, Cone Penetrometer Tests (CPT) commonly supplements SPTs. Both of these tests are performed at the centroid of the vibro-compaction points, thus giving the lowest readings. An average reading could be obtained by testing in the middle of a line connecting two points.

For typical area densification problems, the cost range for a vibro-compaction solution will vary from \$1 to \$3/yd^3 of densified in situ soils, depending mainly on the size of the project, gradation of in situ soils, and degree of densification required. However, in marginal soils where special backfill is required, the costs could be significantly higher, yet the total economics may justify a vibro-compaction solution.

The many factors that can affect the pricing of a vibro-compaction project are listed in Table 4-9.

Table 4-9. Factors Affecting Price of Vibro-compaction Projects

Category	Factors
In situ Material	• Type of Material • In situ Density • In situ Cementation
Backfill Material	• Type • Cost
Densification Requirements	• Load Bearing • Degree of Densification 1. Average relative density 2. Minimum relative density
Project Requirements	• Size • Depth of Densification • Overburden • Type 1. Footing compaction 2. Area compaction • Specifications • Location of Project 1. Labor and union considerations 2. Support equipment availability 3. Weather - freezing weather conditions
Pricing	• Compaction Spacing • Unit Pricing 1. Linear foot 2. Cubic yard

The following procedure may be used for estimating the cost of vibro-compaction:

1. *Determine the performance requirement.* Section 3.3 lists typical requirements for most projects.
2. *Determine the number of compaction points required from the performance requirement*, resulting compaction point spacing, and total project size.
3. *Determine the required depth of compaction from the subsurface investigation and project requirements.*
4. *Cost of vibro-compaction = (# of compactions x depth x unit price) + mobilization.*

5. Price includes supervision, labor, equipment, tools, utilities and backfill added during compaction. (About 1 cubic yard of backfill added for each 5 linear feet of compaction.) Rate of production = 300 linear feet per vibrator per 8 hour day.

Add the cost of additional fill to raise the grade.

3.5.2 Cost Data

Cost information for transportation related vibro-compaction projects, where approximate unit costs are available, is summarized in Table 4-10.

Table 4-10. Cost Information Summary

Pay Item Description	Quantity Range	Unit	Low Unit Price	High Unit Price	Factors Which May Potentially Impact Costs
Mobilization	1	LUMP SUM	$20,000	$30,000	Mobilization cost increases for distances greater than 500 miles. Phased projects may require multiple mobilizations.
Vibrocompaction	Greater than 2,500	LF	$5.00	$9.00	Production rates increase as depth increases. In situ density of soils impacts the average production rate.
Granular Fill Material	–	TON	$7.00	$20.00	Material specifications and haul distance will impact unit costs.

3.6 Case Histories

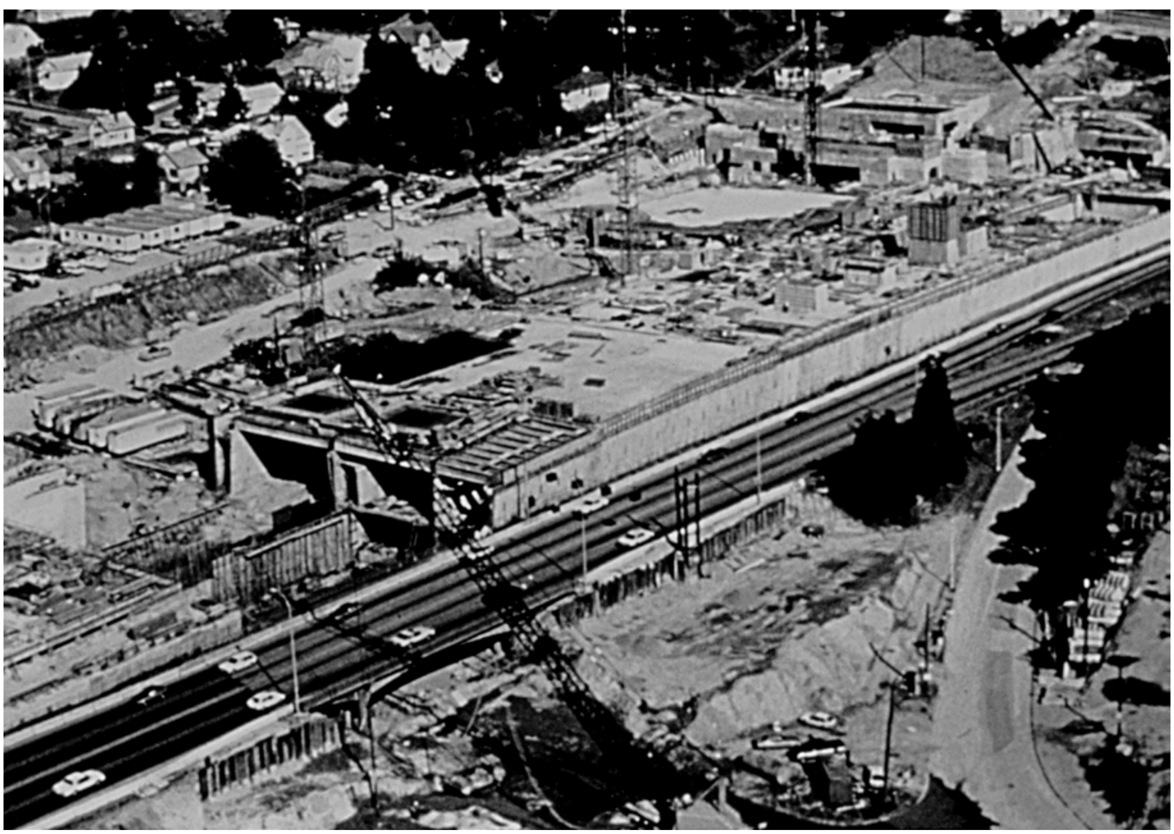
The vibro-compaction technique is used to achieve a variety of design objectives. The case histories selected for this chapter represent different transportation applications.

3.6.1 I-90 Mt. Baker Ridge, Seattle, WA (Hayward Baker 1989)

Environmental considerations played a major role in an extensive improvement and expansion program for the I-90 corridor through the Mt. Baker Ridge area in Seattle, WA. With stretches of the improved interstate designed to carry 50,000 vehicles each way daily, the impact on residential communities was alleviated by deep-cut construction accommodating covered roadways. The roadway structures would support landscaped parks, effectively reclaiming these construction areas.

Massive pier footings on grade were required to support the covered roadway cross-section. In addition to providing vertical support for the cross-section, the footings were also designed to carry the lateral load of embankment soils placed behind the wall. For units 9 and 10 of the 2,590-feet-long roadway section abutting the Mt. Baker Tunnel, the 300-foot-long by 30-foot-wide footing was to be placed directly on soils previously placed for the existing highway embankment. Originally, the footing design assumed a 5,960 psf allowable bearing on to the fill soils. However, subsequent geotechnical investigation determined that the loose to medium-dense, silty, gravelly, fine-to-medium sand fill (approximately 20 foot depth) could not support the 5,960 psf loading without extensive settlement. Washington Department of Transportation engineers, in conjunction with their consultants, considered both deep foundations and in situ soil improvement. Based on time and cost considerations, vibro-compaction was chosen as the best solution.

To meet densification criteria, stone backfill was specified for the vibro-compaction process. At 540 compaction points, a 120 kW vibrator densified the soils to a depth averaging between 15 to 20 feet, as shown in Figure 4-38.



Hayward Baker 1989

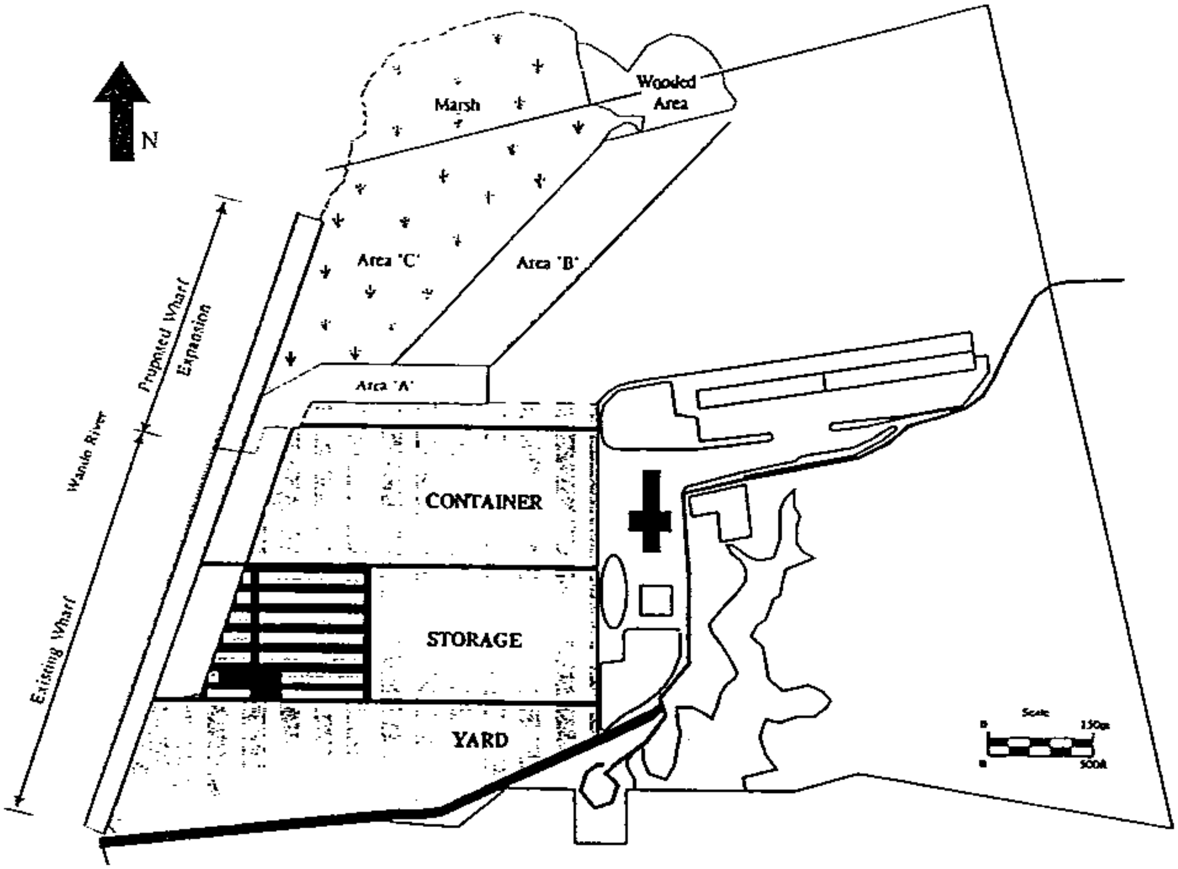
Figure 4-38. Vibro-compaction on Mt. Baker Ridge's Interstate 90.

This treatment depth allowed for densification/reinforcement through the existing soils and through the loose upper zone of the underlying sandy glacial deposit. The area of treatment included a 10 to 20 foot perimeter around the entire footing. The compaction points were spaced on a 6 foot grid pattern with the design intent to limit settlement to a specified requirement of ¾ inch.

Two plate-load tests were performed at selected compaction point locations during the work. A 6 foot by 6 foot plate was placed directly over each of two points and loaded in increments to 210,000 lbf. The total load represented a uniform 5,960 psf pressure on the test plate. The test work indicated that average total settlement under the working design load was approximately (0.5 inch) with permanent plastic deformation upon unloading indicated to be approximately ¼ -inch.

3.6.2 Wando Terminal, Charleston, SC (Hussin and Foshee 1994)

In South Carolina, a site improvement challenge involved the expansion of Wando Terminal, a State port facility in Mount Pleasant, near Charleston. The expanded terminal was designed to serve as a docking facility and as a (267,000 yd^2 (225,000 m^2) concrete-paved container storage yard. The site of the expansion section was located north of the existing facility (Figure 4-39).



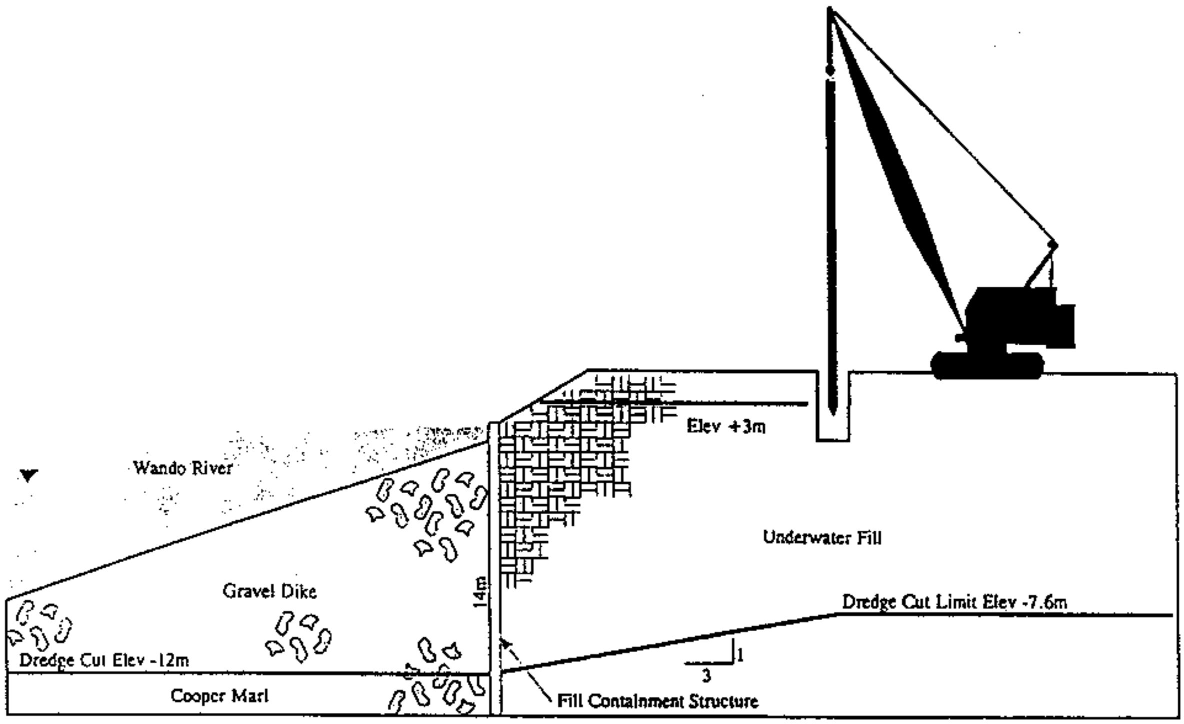
Hussin and Foshee 1994

Figure 4-39. State Pier 41, Wando Terminal.

The storage yard for the expansion was divided into three areas: Area A (53,000 yd^2 (44,500 m^2), Area B (87,000 yd^2 (72,900 m^2), and Area C (131,000 yd^2 (109,300 m^2). (Areas A and B were formerly marshlands that, over 10 years ago, had been filled to elevation +22.0 feet (6.7 m) MLW or higher. The long-term surcharging of these areas had consolidated the underlying marsh deposit sufficiently to eliminate the need for additional ground improvement. However, Area C was composed of virgin marshlands.

Much of the Charleston peninsula is composed of former marshland, filled over the last 350 years with both earthen materials and man-made debris. Many different structures have been built within these areas, and numerous problems have resulted, including areal subsidence in the range of 2 to 4 inches (50-100 mm) per year over the life of a structure. In the early design stages for the new container storage yard, the owner decided that the above settlements could not be tolerated. Since the existing Wando terminal is viewed as the showcase of South Carolina State Port Authority's Charleston facilities, the expansion was required to be of comparable quality. Replacing some, or all, of the deep deposits of marsh mud in Area C with less compressible soil was determined to be the only option.

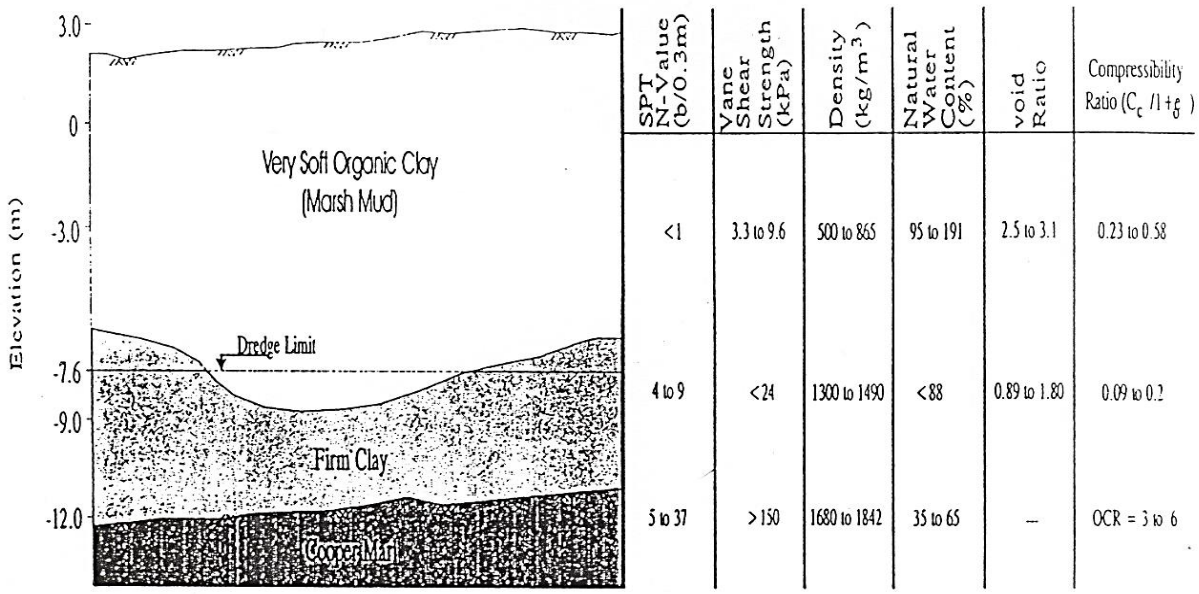
A generalized profile of the subsurface conditions within Area C is depicted in Figure 4-40.



Hussin and Foshee 1994

Figure 4-40. Vibro-compaction at Wando Terminal.

This profile essentially represented the worst-case conditions for analyzing the various ground improvement alternatives being considered to create the container storage yard. As can be inferred from the soil properties listed in Figure 4-41, the marsh mud was extremely soft and compressible. Although still relatively soft, the intermediate "firm" clay was more consistent and did not present the same design challenges with respect to compressibility and stability. The lower stratum (Cooper Marl), due to its high over-consolidation ratio, is virtually incompressible.



Hussin and Foshee 1994

Figure 4-41. Generalized subsurface profile of Area C.

It was determined that to achieve acceptable results, the required foundation improvement program would have to be a three-step process:

1. Dredge the soft clay to elevation -25 feet (-7.5 m) and replacing that material with 1.2 million cubic yards (meters) of clean sand to elevation +10 feet (+3 m).
2. Install vertical drains (wick drains) to accelerate the consolidation of the underlying clays.
3. Transform the very loose sand backfill into dense sand using vibro-compaction (Figure 4-42).

Hussin and Foshee 1994

Figure 4-42. Densification of loose sand backfill during vibro-compaction at Wando Terminal.

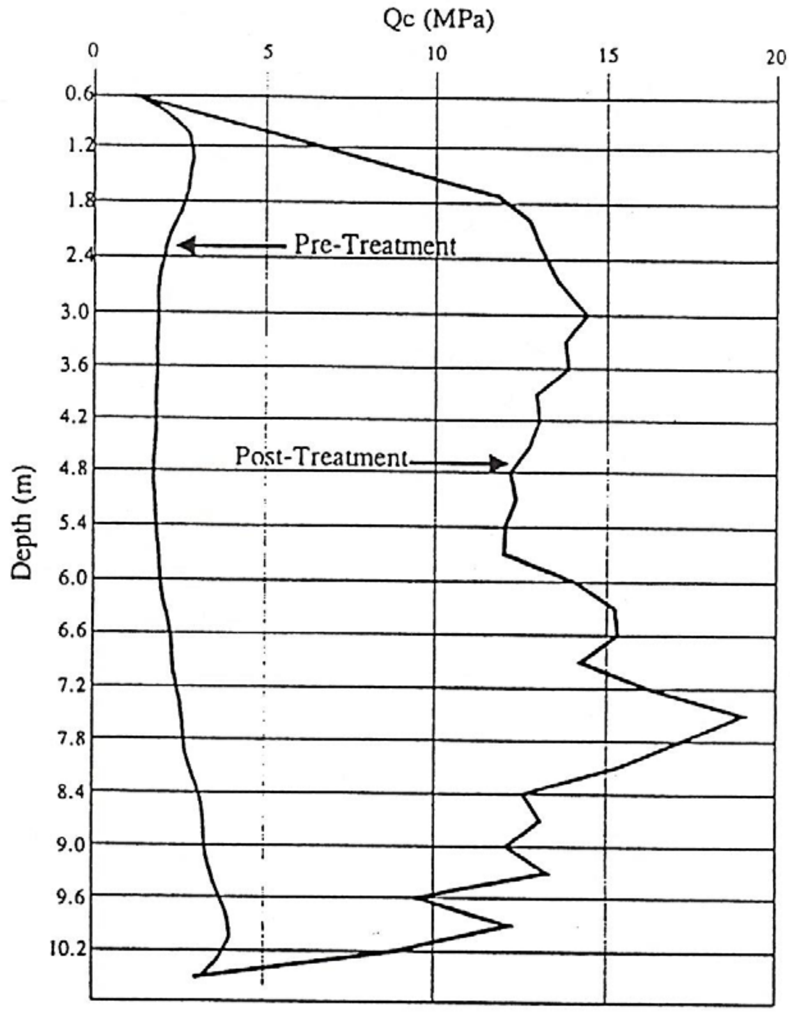
The third step, involving vibro-compaction, would densify the backfill and eliminate a costly intermediate step of dewatering the site.

By selecting a dredge level of -25 feet (-7.5 m) MLW, only isolated pockets of the highly compressible marsh mud would be left in place. By backfilling with clean sand (fines # 5 percent), and inserting wick drains to elevation -40 feet (-12.2 m) MLW on 5 foot (1.5 m) centers, it was estimated that the maximum post-construction settlement of the container yard would include 3.5 to 7 inches (90 to 180 mm) of primary consolidation over the first 2 years, and up to 4 inches (100 mm) of secondary compression over the next 50 years.

Once the specifics of the program were determined, the backfilling of the 130,720 yd^2, (109,300 $m^{2)}$ excavation with underwater fill could begin. This fill was specified to be fine sand with less than 1 percent clay, and less than 5 percent fines (silt and clay), by weight. The contractor elected to fill the excavation by hydraulically pumping the sand from a central dumping area.

With the backfill and wick drains in place, vibro-compaction (utilizing 4 rigs working double shifts, 6 days per week for 5 months) could complete the improvement program (Figure 4-41). At the completion of this process, the sand backfill was completely densified, lowering the surface elevation from 35 feet to 31 feet (10.7 m to 9.5 m).

Pre-treatment and post-treatment cone penetrometer test results (Figure 4-43) confirmed that the required minimum densification had been achieved. Vibro-compaction densification stabilized the soils to support the weight of the containers.



Hussin and Foshee 1994

Figure 4-43. Sample cone penetration results.

Additionally, since the Wando Terminal site is situated within one of the most prominent areas of seismicity along the Atlantic Seaboard, the densification served as a precaution designed to prevent liquefaction of soils should an earthquake occur (Charleston was struck by a large earthquake in 1886).

The uniqueness of this large, multi-step project was twofold. First, its size (130,700 yd^2 (109,300 m^2)) proved to be one of the largest areas treated to date. Second, the program achieved project goals by backfilling the excavation through the water, instead of traditional dewatering and filling the hole with compacted layers of sand. Vibro-compaction proved to be equally effective and considerably more economical than the dewatering alternative.

3.6.3 Manchester Airport, New Hampshire (Sobel et al. 1993)

Construction of a new, 160,000 ft^2 (15,000-square-meter) terminal building at Manchester Airport, New Hampshire, over loose, sandy, potentially liquefiable soils required that a ground improvement program be developed to mitigate the risk of liquefaction during a seismic event.

Design phase borings had revealed delta-deposited, clean, uniformly graded, saturated, fine-to-medium sands from depths of 12 to 45 feet (3.7 to 13.7 m). Laboratory gradation and Standard Penetration Testing revealed the potential for seismically induced liquefaction. The design of the densification program was based on specific parameters developed from

1. Methodology proposed to determine the factor of safety against the occurrence of liquefaction (Seed et al. 1985).
2. Correlation of SPT values to volumetric strain (Tokimatsu and Seed 1987).

Analysis performed in accordance with the above indicated a factor of safety against liquefaction of less than unity under regional design criteria and a volumetric strain of 10 percent of the layer thickness that translated into a potential for 1 foot (0.3 m) of settlement below the building footprint.

Both deep foundation and ground improvement alternatives to allow shallow footing construction were evaluated. The deep foundation alternatives were eliminated due to cost considerations and the uncertainty of performance under liquefaction conditions. Of the ground improvement alternatives considered (including excavation/replacement, dynamic compaction, deep blasting, and compaction grouting), vibro-compaction was selected because of its cost-effectiveness and proven success record in sands.

The vibro-compaction design was required to meet seismic criteria of a design earthquake magnitude of 6.0, a peak ground acceleration of 0.12g, and a minimum factor of safety against liquefaction of 2.0. Allowable differential settlement was determined to be 0.5 inch (12 mm), with a 1 inch (25-mm) allowable total settlement.

To meet these criteria, compaction points were located on a 10 foot by 10 foot (3 m by 3 m) grid. The necessary depths of compaction were determined to be 26 feet (8 m) and 36 feet (11 m). Although design borings had identified potentially liquefiable soils to 45 feet (13.7 m), actual depth-of-treatment selection was based upon performance studies of Japanese sites where liquefaction had occurred.

The spacing and depth of treatment used resulted in the minimum specified relative density where coarse, clean sand was present. Where post-treatment tests indicated that loose relative density conditions remained, the spacing was reduced.

The project required over 2,600 compaction points. Thirty-four post-treatment SPTs were conducted, typically at 1100 yd^2 (900 m^2) intervals, at the centroid of the compaction point grid to assess the vibro program. Compliance with project specifications was generally achieved after initial treatment. Where SPT values were at or slightly below specified values, at depths ranging from 12 feet to 17 feet (3.7 m to 5.2 m), it was attributed to the presence of a dense crust of coarse sand temporarily arching over the loose material below. Subsequent testing, after a waiting period of 1 to 3 weeks, showed that, in most instances, N values had increased with time to meet the specified criteria.

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Chapter 5

AGGREGATE COLUMNS

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1.0 INTRODUCTION

1.1 Description and History

Over the past 30 years, aggregate column technology has become established in the United States as a viable ground modification technique. It has been applied extensively for remediation and new construction of transportation facilities. Construction of highway embankments using conventional design methods, such as preloading, dredging, and soil displacement techniques, can often no longer be used due to environmental restrictions and post-construction maintenance expenses. Aggregate columns have a proven record of experience and are ideally suited for use in clays, silts, loose silty sands, and uncompacted fills. The history and development of aggregate columns, the focus and scope of this technical summary and primary references are discussed in this section.

1.1.1 Description

This technical summary on aggregate columns includes both rammed aggregate piers and stone columns. The similarities and differences of both types of columns will be presented in the following sections of the document. When discussing the attribute of both stone columns and rammed aggregate piers the term aggregate columns will be used. However, if an attribute is specific to only one of the two types of aggregate columns, the specific column will be identified as a stone column or rammed aggregate pier.

1.1.2 Stone Columns

Stone column construction is accomplished by down-hole vibratory methods. The technique of creating stone columns involves the introduction of backfill material into the soil so that dense and sometimes deep columns of aggregate are formed that are tightly interlocked with the surrounding soil.

The stone column construction technique is known as either vibro-replacement or vibro-displacement, as follows:

- **Vibro-replacement** - Generally refers to the wet, top feed process in which jetting water is used to aid the penetration of the ground by the vibrator. Due to the jetting action, part of the in situ soil is washed to the surface. This soil is then replaced by the backfill material (e.g. stone). A dry feed process, with soils requiring predrilling, will also result in in situ soil being brought to surface.
- **Vibro-displacement** - Generally refers to the dry, top or bottom feed process; almost no in situ soil appears at the surface, but is displaced by the backfill material.

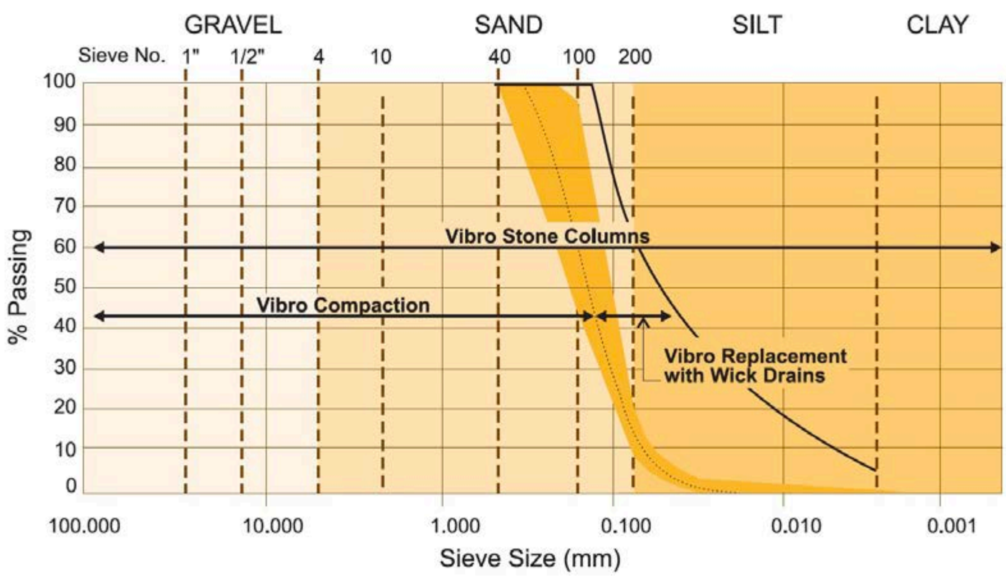
The product of both the vibro-replacement and vibro-displacement construction methods is generically referred to as a stone column.

1.1.3 Rammed Aggregate Piers

Rammed aggregate piers are installed by drilling 18- to 36-inch diameter holes into the foundation soils and ramming lifts of well-graded aggregate within the holes to form stiff, high-density aggregate columns. The drilled holes typically extend from 7 to 33 feet below grade. The first lift of aggregate forms a bulb below the bottoms of the piers. Subsequent lifts of aggregate are typically 12 inches in thickness. Ramming takes place with a high-energy beveled tamper that both densifies the aggregate and forces the aggregate laterally into the sidewalls of the hole. This action increases the lateral stress in the surrounding soil, further stiffening the stabilized composite soil mass.

1.2 Historical Overview

For over 50 years, deep vibrators have been used to improve the bearing resistance and settlement performance of weak soils. As early as 1936, methods and equipment were developed that enabled the compaction of non-cohesive soils to depths of 60 feet with excellent results. This original process is now referred to as vibro-compaction or vibro-flotation. Stone-column technology developed as a natural progression from vibro-compaction and extended vibro-system applications beyond the relatively narrow application of densification of clean, granular soils, as shown in Figure 5-1.



Courtesy Hayward Baker

Figure 5-1. Soils applicable for stone columns.

The ability to compact soil depends mainly on the grain size distribution of the soil. Soils with grain size distribution curves lying entirely on the coarse side of the hatched line in Figure 5-1 are generally readily compacted by the vibro-compaction process. If the grain size distribution curve falls to the right of the dotted line the soil is not readily compacted by vibro-compaction. It is for these types of soil and their related problems that necessitated the development of stone column technology.

It is interesting to note that one of the first documented uses of stone columns was for the Taj Mahal in India, completed in 1653. This historic structure has been successfully supported for more than three centuries by hand-dug pits backfilled with stone. The concept of stone columns was also used in France in the 1830s to improve native soil (FHWA 1983). Modern techniques were first implemented during the 1960s in Europe. After extensive use in Europe, the stone column technique was introduced in the United States in the 1970s, but saw limited use in its first 12 years, with only 21 completed projects. However, by 1994, this number had increased to over 400. This growth is due to the better understanding of the design concepts and economics of stone column techniques and the fact that more projects were being built on sites with poor soil. Today stone columns are used extensively to improve the bearing resistance of soft compressible soils.

Rammed aggregate piers were developed in the United States in 1984. The concept of short dug pits backfilled with aggregate to support structures (i.e., rammed aggregate piers) is not new and has been previously documented in the literature. Refinements and improvement to this basic technique have been introduced in the last 25 years under the trade name Geopier®. These rammed aggregate piers are a ground improvement system that is used extensively to improve the bearing resistance of foundation soils. Rammed aggregate piers are installed by drilling 2 to 3 foot diameter holes into the foundation soils and ramming lifts of either well-graded or open graded aggregate within the holes to form very stiff, high-density aggregate piers. The drilled holes typically extend from 7 to 33 feet below grade.

1.3 Focus and Scope

The focus and scope of this technical summary on aggregate columns is to provide guidance on: applications where the technology can be utilized, design, contracting methods, and quality assurance. References are cited where more detailed technical information can be obtained, and typical costs are given in order to make a preliminary technical and economic evaluation as to whether aggregate columns, and related technologies, are appropriate for a given site and application. It is the intent of this document to serve as a reference on aggregate columns and how they may be best utilized on a ground modification project by discussing their construction, utilization, and limitations.

1.4 Primary References

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1.5 Related Technologies

There are a variety of column technologies that are related, and similar, to aggregate columns. Many of these are proprietary technologies developed by ground modification contractors. Some are equipment and installation variations, and may be more suited to specific installation conditions, such as beneath the water table or in very soft soils. Many of these related technologies use Portland cement binder with the aggregate and, thus, a more rigid (cemented aggregate) column is constructed. Another cement based column option is to use concrete for construction of the columns. Two common cement based columns are vibro-concrete columns (VCCs) and controlled modulus columns (CMCs); these are briefly discussed below. Cement based, concrete columns may be used in softer soils, without casing, and can be used to produce a stiffer element than aggregate columns.

1.5.1 Vibro-Concrete Columns

Vibro-Concrete Columns (VCC) are considered a sister technology to stone columns, with concrete replacing the stone in the column. The vibro-concrete column is a non-proprietary process that employs a vibro-displacement (i.e., bottom feed) depth vibrator to penetrate the soils to a level suitable for bearing. Concrete is pumped through the vibrator assembly during initial withdrawal. The vibrator then re-penetrates the concrete, displacing it into the surrounding soil to form an enlarged column base. The vibrator is then slowly withdrawn as concrete is pumped at a maintained pressure to form a continuous shaft of concrete up to ground level. At ground level, a slight mushrooming of the concrete column is constructed to assist the transfer of the applied load to the vibro-concrete column.

The vibro-concrete column was first developed in Europe in 1976. Since stone columns derive their strength and settlement characteristics from the surrounding soil, their capacities are significantly reduced in very soft clay or peat with a thickness greater than 1 to 2 times the diameter of the column. Vibro-concrete columns were developed to treat these soils. Instead of feeding stone to the tip of the vibrator, concrete is pumped through an auxiliary tube to the bottom of the hole. This method can offer the ground modification advantages of the vibro-systems, with the load carrying characteristics of a deep foundation. The first installation of vibro-concrete columns in the United States was in 1994 in Pennsylvania, and was used in support of an oil storage tank. They have been used to support embankments over soft organic soils in many states including Florida, Illinois, Maryland, New Jersey, Puerto Rico, and South Carolina.

See http://www.GeoTechTools.org for detailed information and guidance on VCCs as well as for other cement based column technologies.

1.5.2 Controlled Modulus Columns

Controlled modulus columns (CMCs) are produced with a proprietary process and are similar to VCCs in that the final product is a concrete column. CMCs are constructed using a reverse auger method where the auger displaces the site soil until an adequate bearing layer is reached. The auger has a hollow stem through which a low slump concrete is pumped as the auger is withdrawn. CMCs are a patented technology.

The CMC technique was developed in the early 90's in France by the Menard Group. Menard developed a series of specifically designed machines and tooling that enabled the technology to rapidly grow in use in Europe. DGI-Menard introduced the technology in 2003 in the USA and Canada with the first CMC project in Vermont for the support of a home improvement store.

1.5.3 Design and Construction Considerations of VCCs and CMCs

A generalized summary of the factors affecting the feasibility of stabilizing soft ground with VCCs and CMCs follows:

1. The allowable design load for VCCs and CMCs is a function of the diameter of the column, the allowable strength of the concrete, and the strength of the bearing layer. Typical column diameters range from 18 to 24 inches for VCCs and 10 to 18 inches for CMCs. Typical allowable design loads range from 150 to 250 kips for VCCs and 75 to 150 kips for CMCs.
2. VCCs and CMCs are typically used in very soft clay and organic soils.
3. Typical lengths vary from 20 to 75 feet.

See http://www.GeoTechTools.org for information on design, specification, quality assurance and construction of VCCs. This information can also provide some guidance for other cement based column technologies.

2.0 DESIGN CONSIDERATIONS

Aggregate column construction involves the partial replacement or displacement of unsuitable subsurface soils with a vertical column of compacted aggregate. This section discusses applications, advantages and disadvantages, and design considerations for aggregate columns.

2.1 Applications

Aggregate columns can be applied to increase bearing resistance, reduce total and differential settlements, accelerate the time rate of settlement, improve slope stability, and reduce the liquefaction potential of soil. Typical applications include foundation improvement for the construction of highways, embankments, warehouses, and light industrial buildings.

2.1.1 Embankments

One typical application of aggregate column technology is the stabilization of large area loads such as highway embankments. The use of aggregate columns offers a practical alternative, where conventional embankments cannot be constructed due to stability considerations. Applications include moderate-to-high fills on soft soils, fills that may be contained by mechanically stabilized earth, and construction on slopes where stability cannot otherwise be obtained. An important related highway application is slope stabilization. In 1987, the Soil Mechanics Bureau, New York State DOT, reported on the use of the dry bottom feed vibro-displacement method to solve a slide problem (Sung and Ramsey 1988).

A considerable amount of highway widening and reconstruction work has occurred over the last several decades. Some of this work involved building additional lanes immediately adjacent to existing highways constructed on moderate-to-high fills over soft cohesive soils, such as those found in wetland areas. For this application, differential settlement between the existing and new construction is an important consideration, in addition to embankment stability. Support of these new fills on aggregate columns offers a viable design alternative to conventional construction.

2.1.2 Bridge Approach Fills

Aggregate columns can be used to support bridge approach fills, to provide stability, and to reduce the costly maintenance problem from settlement at the joint between the approach fill and bridge. In 1989, the Texas DOT used 13,000 lineal feet of aggregate columns to support mechanically stabilized earth walls for the U.S. 77 overpass situated in Brownsville. In 1990, the Texas DOT utilized 42,000 lineal feet of 13 to 20 foot long aggregate columns for Brownsville Road over U.S. 77.

Aggregate column supported embankments can be constructed to greater heights than conventional approach embankments over soft foundation soils. Therefore, the potential exists to reduce the length of bridge structures by extending the approach fills supported on aggregate columns. Embankment fills can be placed faster due to the combined effects of accelerated drainage and consolidation, and the increase in shear strength supplied by aggregate columns.

2.1.3 Bridge Abutment and Foundation Support

Aggregate columns can be used to support bridge abutments at sites that are not capable of supporting abutments on conventional shallow foundations. At such sites, an important additional application involves the use of mechanically stabilized earth walls supported on stone columns.

Another potentially cost effective alternative to pile foundations for unfavorable site conditions is to support single span bridges, their abutments, and their approach fills on aggregate columns. This technique minimizes the differential settlement between the bridge and approach fill.

2.1.4 Liquefaction

In earthquake prone areas, aggregate columns can be used to reduce the liquefaction potential of cohesionless soils supporting embankments, abutments, and soils beneath shallow foundations. Aggregate columns can also be used to reduce the liquefaction potential of cohesionless soils surrounding existing or proposed pile foundations. This application has been used quite extensively for major bridges on pile foundations through liquefiable soils in the Pacific Northwest.

2.2 Advantages and Potential Disadvantages of Aggregate Columns

2.2.1 Advantages

Aggregate columns are a technical and potentially economical alternative to deep foundations, capable of improving the soil sufficiently to allow less expensive, shallow-foundation construction. Aggregate columns are also more economical than the removal and replacement of deep, poor bearing soils, particularly on larger sites where the groundwater is close to the surface. Where the infrastructure precludes high-vibration techniques, such as conventional pile driving, dynamic compaction or deep blasting, the low-vibration aggregate column technique is often viable. If time is critical to project start-up, site modification by aggregate column installation can be achieved quicker than by pre-loading the soils. In seismic areas, aggregate columns can reduce dynamics settlements to acceptable levels, and in some cases may densify the soils beyond the threshold of liquefaction. Aggregate columns also provide radial drainage and a vertical drainage path for excess pore water pressure dissipation when low fines content aggregate is used, as well as densifying the liquefiable soils.

2.2.2 Disadvantages

Aggregate columns are not a solution for all soft soil problems. Strata of peat and other organic materials, and very soft clays with a thickness greater than the diameter of the aggregate column can be inappropriate for aggregate column construction, as they offer inadequate lateral support to effectively create the column or to ensure long-term performance. Dense overburden, boulders, cobbles, or other obstructions may require pre-drilling prior to installation of stone columns (rammed aggregate piers that auger a hole to create the column generally can overcome this disadvantage).

Cost, when compared to other solutions, can be a disadvantage of aggregate columns. The need to channel and dispose of spoil water in wet feed construction and lateral ground displacement with a dry construction process may be major disadvantages at some locations. Removal of spoil from the rammed aggregate pier, or predrilled stone column, installation may be a major disadvantage when contaminated soils are present. Rammed aggregate piers are more costly to install when casing is required, and when casing is used this technology may not densify granular soils as effectively as stone columns.

2.3 Feasibility Evaluations

The aggregate column technique of ground modification has been successful in: (1) improving stability of both embankments and natural slopes; (2) increasing bearing resistance; (3) reducing total and differential settlements; (4) reducing the liquefaction potential of cohesionless soils; and (5) increasing the time rate of settlement.

2.3.1 Geotechnical

The degree of densification resulting from the installation of vibro systems is a function of soil type, silt and clay content, soil plasticity, pre-densification relative densities, vibrator type, stone shape and durability, aggregate column area, column spacing, and energy applied. Experience has shown that soils with less than 15 percent passing a #200 sieve, and clay contents of less than 2 percent will densify due to vibrations. Clayey soils do not react favorably to the vibrations, and the improvement in these soils is measured by the percent of soil replaced and/or displaced by the aggregate column. In the case of clayey soils, the ground improvement is achieved by reinforcing the soil.

A generalized summary of the factors affecting the feasibility of stabilizing soft ground with aggregate columns is as follows:

1. The allowable design loading of an aggregate column should be relatively uniform and is limited by the lateral support the in situ soil can develop. Typically, with good lateral support, a maximum of 110 kips per column is used; and typically, the composite factored bearing resistance is increased to 2,000 to 8,000 psf.
2. The most significant improvement is likely to be obtained in compressible silts and clays ranging in shear strength from 300 to 1000 psf.
3. Aggregate columns should not be used in highly sensitive soils. Special care must be taken when using aggregate columns in soils containing organics and peat lenses or layers with undrained shear strength of less than 300 psf. Because of the high compressibility and low strength of these materials, little lateral support may be developed and large vertical deflections of the columns may result. When the thickness of the organic layer is greater than one to two aggregate column diameters, the ability to develop consistent column diameters becomes questionable.
4. Ground improved with stone columns reduces settlements typically by 50 to 70 percent of the unimproved ground response and differential settlements from 5 to 15 percent of unimproved soil response. Ground improvement with rammed aggregate piers can reduce settlement to less than 1 inch, in some loading and subsurface conditions.
5. Due to the development of excessive resistance to penetration of the vibrator a practical upper limit is in the range of an undrained strength of 1000 to 2000 psf for stone columns. If stone columns are used in these stiff soils or through stiff lenses, the column hole is commonly pre-bored, which is often the case in landslide projects. This may result in a significant additional cost.
6. The installation of rammed aggregate piers using the typical replacement method (drilled method) in soils that do not stand open during drilling (i.e., loose granular soils, very soft cohesive soils) may require the use of temporary casing, which reduces the installation rate and increases the cost of the piers.
7. Typically, the maximum practical depth of stone columns and rammed aggregate piers is 100 feet and 35 feet respectively.

2.3.2 Environmental Considerations

The selection of the most appropriate aggregate column installation method should consider the environmental effects of the installation. Soil spoils must be contained, particularly fines from air or water jetting operations. The designer may select an alternate column system that does not replace the in situ soils.

2.3.3 Site Consideration

Site conditions should always be considered when selecting a ground modification technology. The installation of aggregate columns requires sufficient headroom (typically 8 to 10 feet more than the depth of penetration of the column) for the construction equipment. Adjacent buildings and structures must be monitored for heave when using vibro-displacement stone columns.

2.4 Limitations

The major limitation for aggregate columns is that they are not appropriate in very soft and sensitive fine-grained soils and organics. Stone may not be readily available near the project site, leading to potentially significant cost ramifications. Rammed aggregate piers have the additional limitation on the depth of the column (i.e., typically 35 feet).

2.5 Alternative Modification Methods

The following alternative methods, which are similar in concept to aggregate piers, have been used.

2.5.1 Gravel Drains

In Japan, gravel drains are installed by backfilling inside a casing and densifying the stone with an interior vibrator as the casing is extracted. This provides a good drain, but does little to densify the soil outside the casing. For soils with a high liquefaction potential, gravel drains alone may not be able to handle the excess pore pressures, and liquefaction may still occur.

2.5.2 Sand Compaction Piles

This system is also used extensively in Japan. Sand compaction piles are constructed by using a vibratory hammer to install a steel casing to the desired elevation. The casing is filled with sand as it is extracted. For more information see *The Sand Compaction Pile Method* (Kitazume 2005).

2.5.3 Rammed Stone Columns

In Belgium, rammed stone columns have been constructed by driving a casing, placing granular backfill and dropping a heavy weight on the stone as the casing is extracted. While this system can create some compaction of the surrounding soil, it is a very slow process (250 ft/shift/rig) and, therefore, not economically competitive.

The following alternative methods are covered in the other chapters of this manual:

- Prefabricated vertical drains either with or without preloading – Chapter 2
- Deep and mass mixing methods – Chapter 7
- Jetted grout columns – Chapter 8

3.0 CONSTRUCTION MATERIALS AND EQUIPMENT

3.1 Stone Columns

3.1.1 Construction

The primary methods of constructing stone columns are vibro-replacement (wet, top feed) and vibro-displacement (dry, top or bottom feed). Where environmental concerns are strong, the dry process will typically be required. However, the wet process is more economical, if environmental concerns are not as relevant to the project. These processes are illustrated in Figures 5-2 and 5-3.

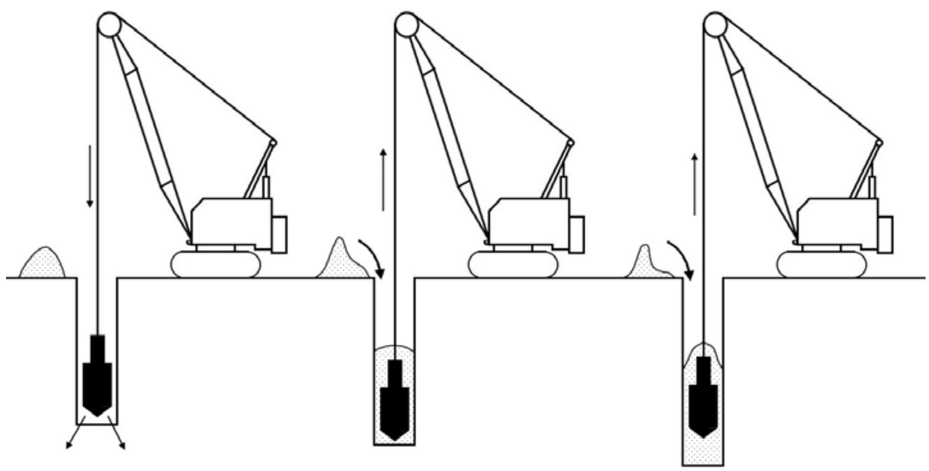


Figure 5-2. Top feed vibro-replacement.

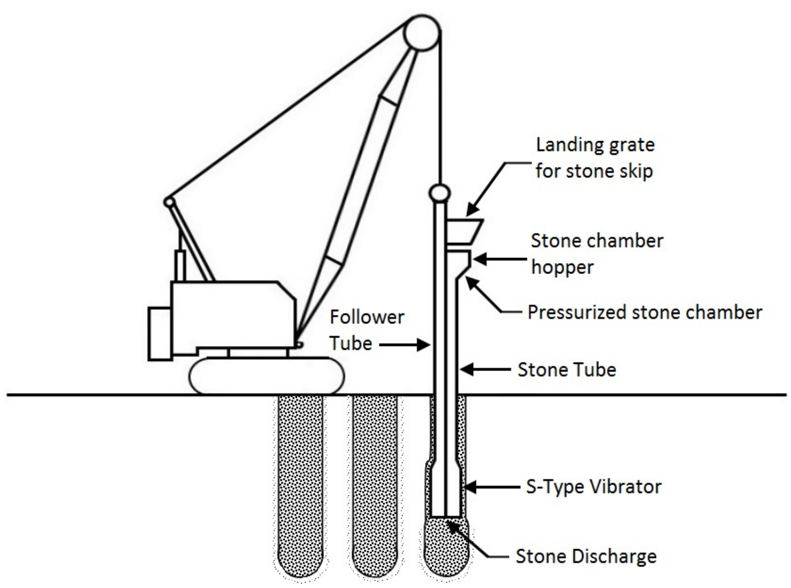


Figure 5-3. Bottom feed vibro-displacement.

3.1.1.1 Vibro-Replacement (Wet, Top Feed)

The original stone column installation technique, called vibro-replacement or the wet process, utilizes a high-pressure jet of water to open a hole that the probe follows into the ground. The probe is then retracted in increments, and stone is introduced into the void from the surface (Figure 5-2). After every increment, the probe is lowered into the new column material, thereby densifying and compacting the stone column and, potentially, the surrounding soil (depending on percent fine content). This method is best suited for sites with soft to firm soils with undrained shear strengths of 300 to 1000 psf and a high groundwater table.

3.1.1.2 Vibro-Displacement (Dry, Top and Bottom Feed)

As the jetting water effluent from the vibro-replacement method includes the finer portion of the in situ soil, environmental problems encountered in containment, removal, and disposal of the effluent had to be addressed. To resolve these problems, the dry top and dry bottom feed techniques were developed. Using the oscillations of the vibrator coupled with its deadweight, air jetting, and/or pre-augering, the vibrator is inserted into the ground without the use of jetting water. For shorter stone columns, the stone can still be fed into the annulus created by the vibrator from the surface, as shown in Figure 5-2. For deeper treatment or where the hole may collapse, the stone is fed to the bottom of the vibrator through an attached tremie tube as shown in Figure 5-3. The first major use of the dry bottom feed vibro-displacement system in the United States was for the Steel Creek Dam foundation at the Department of Energy's Savannah River Plant, South Carolina, in 1985 (Dobson 1987).

3.1.1.3 Equipment

The equipment used to form stone columns is comprised of the following:

- Vibrator, which is suspended from extension tubes with air or water jetting systems
- Crane or base machine, which supports the vibrator and extension tubes
- Stone delivery system
- Control and verification devices

The principal piece of equipment used to achieve compaction is the vibrator, which ranges in diameter from 12 to 16 inches and in length from approximately 10 to 16 feet. A suspended vibrator is shown in Figure 5-4, and a cross section of a typical vibrator is shown in Figure 5-5.

Figure 5-4. Suspended vibrator.

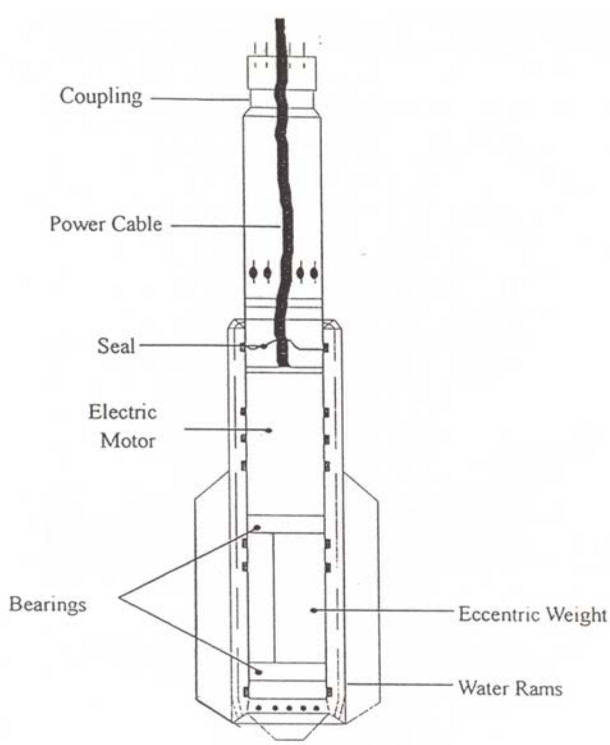


Figure 5-5. Typical vibrator cross-section.

Horizontal vibrations are produced close to the base of the vibrator and are induced by rotating eccentric weights mounted on a shaft and driven by a motor located in the upper part of the vibrator casing. Both electric and hydraulic power can be used to power the motor. Early units were driven by motors in the 22 to 60 kW range, but more recent machines develop up to 125 kW. Centrifugal forces of up to 60 kips at frequencies varying from 1200 to 3000 rpm are currently achieved. Abrasion resistant wear plates are added to the sides of the vibrator, protecting it from excessive wear during raising and lowering from the ground. Fins located on the sides of the vibrator reduce rotation. Follower or extension tubes, typically of a similar or smaller diameter to the vibrator unit, are attached to it and allow treatment of soils at depth. An elastic coupling is used to isolate the vibrator from the extension tubes and to prevent vibrations from traveling up the extension tubes to the supporting crane or base machines.

Water or air can be conveyed to the top of the extension tubes by flexible hoses and, subsequently, through the extension tubes to the vibrator. The water or air is generally fed to the nose of the vibrator to assist penetration into the soil. The thickness of soil to be treated determines the overall length of vibrator, extension tubes, and lifting equipment, which, in turn, determines the size of crane to be used. The vibrator is suspended from the boom of a crane; a 33-foot probe can be easily handled using a 40 ton crane with a 40-foot boom. Penetration of the probe is accomplished by vibration, jetting media (air or water), and dead weight. The greater the depth of soil to be treated, the larger the required crane.

The construction of stone columns requires the importation and handling of substantial quantities of granular material. The granular material is routinely handled with front end loaders, working from a stone pile and delivering stone to each stone column location. For the top feed method, the stone is end-dumped into the hole created by the vibrator. For the bottom feed system, stone is fed into a skip. The skip can then supply pipes in the vibrator and extension tube assembly with stone. The pipes lead to the vibrator nose and, during operations, stone is fed continuously to the very point of compaction. Vibrators can be retained in the ground during compaction work, thus maintaining the hole in an open condition, and enabling a high integrity stone column to be constructed. Alternate stone transport systems have been developed, which allows the transport of stone backfill through a 6-inch hose instead of a skip, along a leader. Typical equipment to install stone columns is illustrated in Figures 5-6 through 5-9.

Figure 5-6. Truck mounted crane utilized for top feed vibro-replacement.

Courtesy Hayward Baker

Figure 5-7. Stone column dry bottom feed rig.

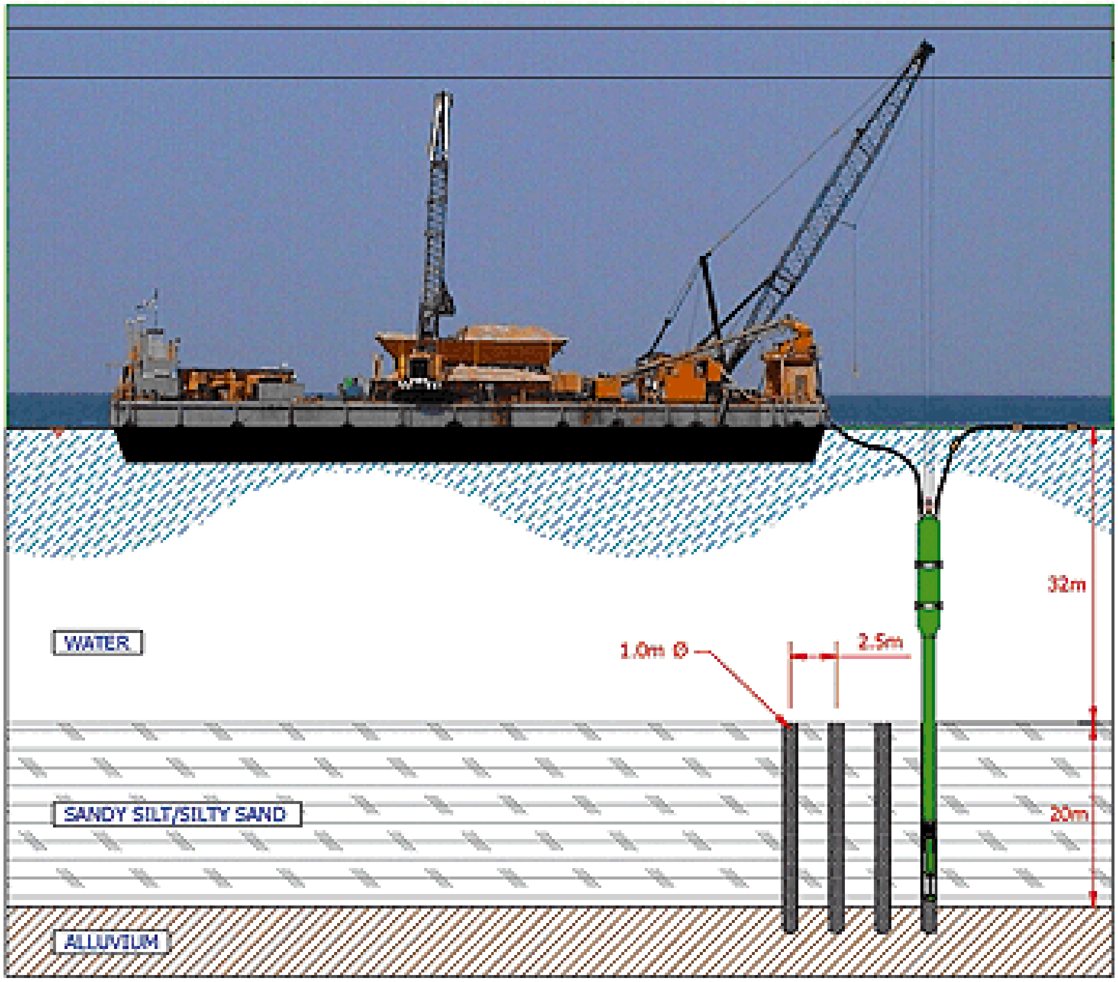
Courtesy Treviicos

Figure 5-8. Dry bottom feed rig.

Courtesy Subsurface Constructors

Figure 5-9. Top feed vibro rig.

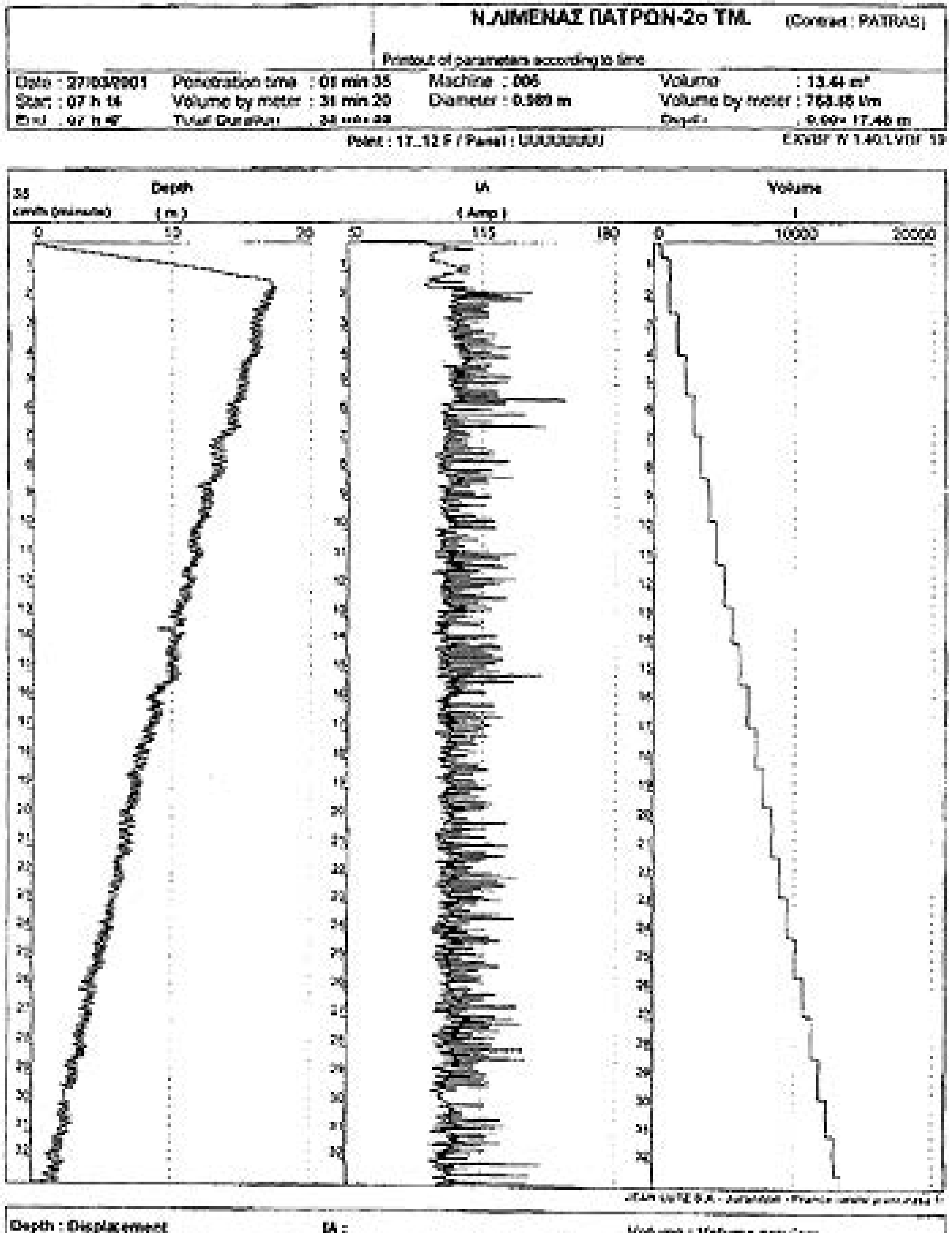
A method developed almost 20 years ago, the marine double-lock gravel pump technique (a patented process), has been developed to deliver stone to the bottom of the vibrator in underwater applications. This method transports the gravel through a system of hoses using air pressure supplied through an air compressor. The double-lock system provides excess air pressure at the tip of the vibroprobe at all times. This minimizes the potential for soil intrusion into the discharge pipe. This method is illustrated in Figure 5-10.



Vibroflotation Group

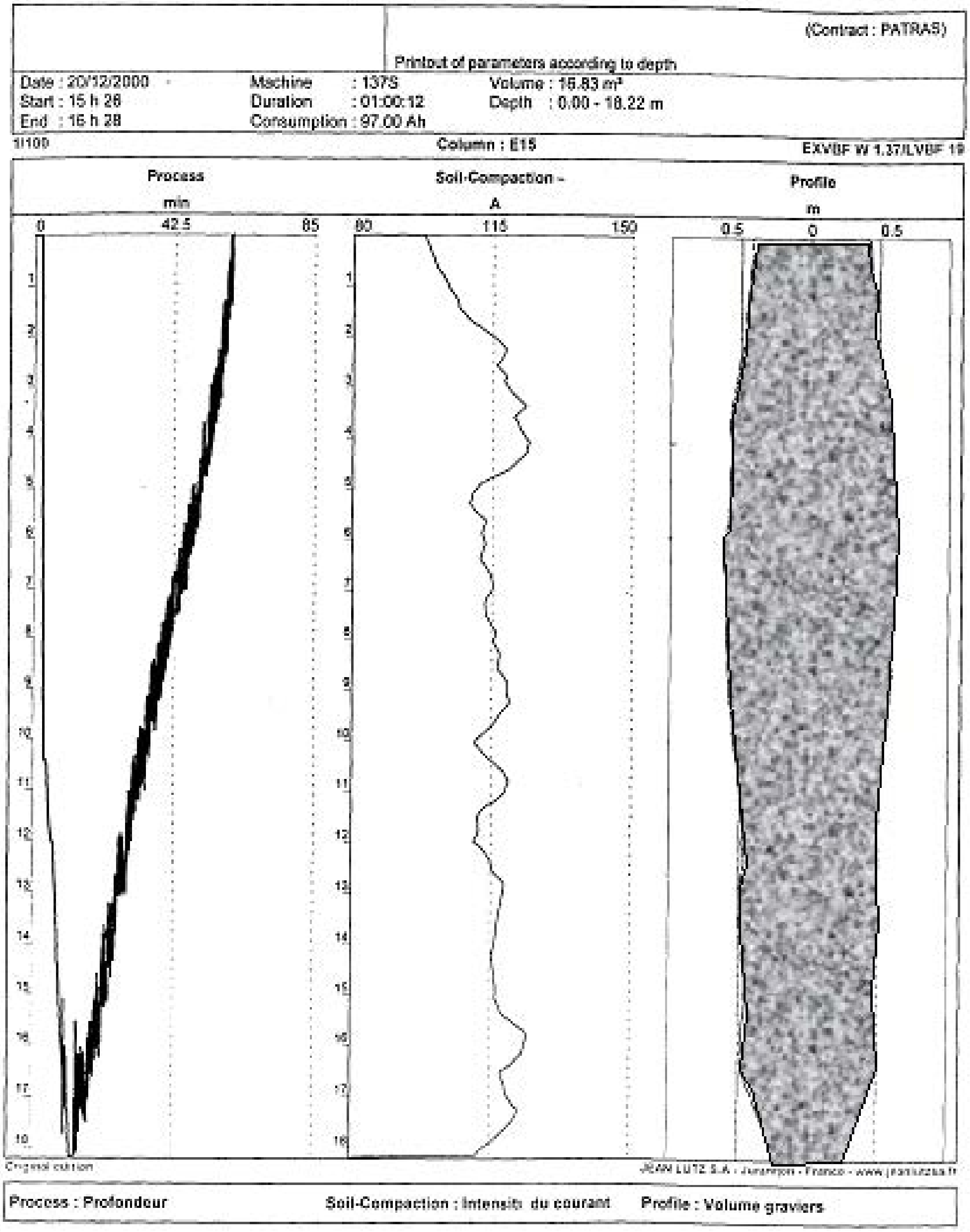
Figure 5-10. Marine double-lock gravel pump.

Instrumentation packages that provide a continuous record of construction data for each stone column are now common. Measurements of depth, power consumption, and stone consumption are recorded against time and provided on a printout at the time of construction. Such instrumentation is available for bottom feed vibrator systems and has been used in Europe since the 1980s and in the United States since 1993. Sample output is shown in Figures 5-11 and 5-12.



Vibroflotation Group

Figure 5-11. Quality control output over time for dry bottom feed vibro-displacement: depth (left), amperage (middle) and gravel (right).



Vibroflotation Group

Figure 5-12. Quality control output over depth for dry bottom feed vibro-displacement: amperage (left), compaction (middle), and column diameter over depth (right).

3.1.2 Backfill Material

The size, gradation, and shape of backfill for stone columns usually depends upon the:

- construction technique used,
- subsoil properties that the column is being constructed in,
- application for which the columns are being designed for, and
- local availability of materials.

Backfill may vary by method or technique of placement, i.e., top feed or bottom feed, jetting or not, water or air jetting, etc. The method of placement is also a function of the subsoils characteristics. Furthermore, the application has to be considered in the selection of the backfill. For example, drainage characteristics are crucial in liquefaction prevention and shear strength is critical in slope stabilization projects. Whereas, drainage and shear strength properties are generally not critical for increasing bearing resistance applications and, of course, local availability and material costs will factor into column backfill selection.

For vibro-replacement stone columns, subangular or angular gravel of nearly uniform grading 1.0- to 2.5-inch in size is often used. This size backfill passes easily around the vibrating probe, while it is still in the hole. The larger sized in situ material suspended in the water usually fills the voids between the stone resulting in a rigid column.

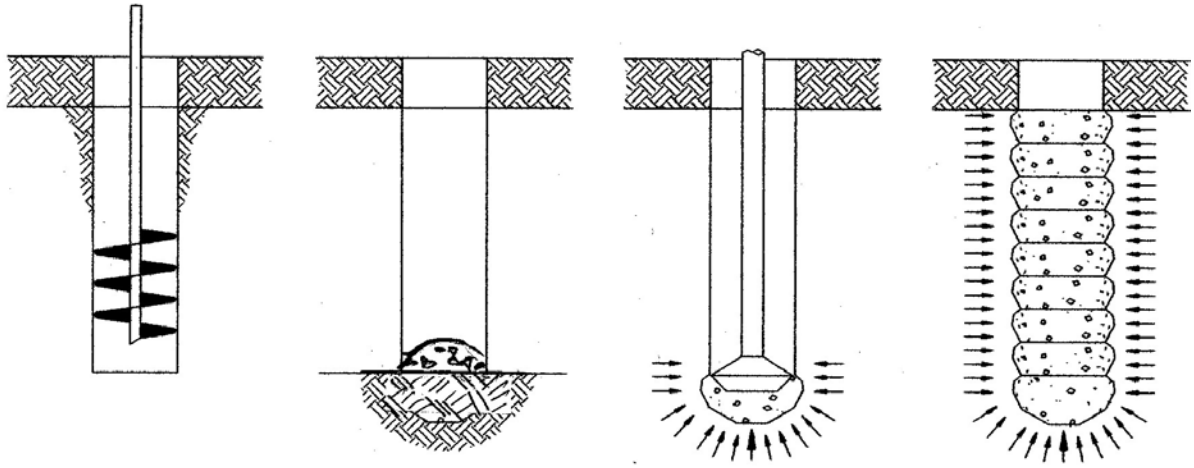
An important factor in the successful construction of wet stone columns is keeping the flushing water flowing at all times to wash out the soil fines that infiltrate the stone and to aid in stabilizing the hole. Keeping the probe in the hole at all times during installation increases the stability of the jetted hole.

For vibro-displacement, well graded backfill with a gradation from 0.4- to 3-inch or up to 4-inch is generally used to achieve mechanical interlock and filling of voids. The finer backfill sizes are included to provide an intermediate particle size between the in situ clay and gravel. The bottom feed method is restricted to aggregates of approximately 0.4- to 1.4-inch in size to avoid blockage of the equipment.

3.2 Rammed Aggregate Columns

3.2.1 Construction

The primary method of constructing rammed aggregate piers is the replacement method, shown in Figure 5-13.



Courtesy Geopier Foundation Company

Figure 5-13. Replacement method, rammed aggregate pier construction process: from left to right, (1) make a cavity, (2) place stone at bottom of cavity, (3) ram stone to form bottom bulb, (4) place and ram thin lifts to form undulated side shaft.

The replacement method consists of the following:

- Excavate pier to design depth (make cavity), use casing if hole will not stay open
- Place open graded stone at bottom of cavity, in a 24-inch thick lift
- Ram stone at bottom of cavity
- Place and ram 12-inch lifts of stone until the elevation of the top of the column is achieved.

Rammed aggregate pier construction equipment is shown in Figures 5-14 and 5-15.

Courtesy Geopier Foundation Company

Figure 5-14. Rammed aggregate pier tamper.

Courtesy Geopier Foundation Company

Figure 5-15. Replacement method, rammed aggregate pier with predrilling.

The construction equipment consists of three pieces of equipment: an excavator-mounted drill, an excavator-mounted hammer, and a skid-steer loader. The excavator-mounted drill is a conventional excavator with typically 2- to 3-foot diameter drilling tools. Common excavators are generally used to minimize problems and costs associated with transportation of large construction equipment. The excavator-mounted tamper is a conventional excavator with a modified concrete breaker attached to the machine that is capable of delivering 245 to 650 kip-lbf per 1 foot per lift of energy for tamping, that both densifies the aggregate and forces the aggregate laterally into the sidewalls of the hole. This action increases the lateral stress in the surrounding soil. A composite alloy shaft with an attached beveled-hammer is connected to the modified concrete breaker. The size of the tamper should be at least 85% of the plan area of the cavity. The third piece of equipment is a skid-steer loader that delivers the aggregate to the hole. Most skid-steer loaders are track-driven to provide stability and better maneuverability on muddy sites.

3.2.2 Backfill Material

For rammed aggregate pier construction, clean 1- to 3-inch stone is commonly specified for the bottom bulb. The same material is used throughout the pier if radial drainage of the pier is included in the design solution; otherwise, a well-graded base course aggregate is used.

4.0 DESIGN

4.1 Stone Columns

4.1.1 Design Considerations

Although the method of introducing the backfill material, and gradation of backfill, is somewhat different for vibro-replacement and vibro-displacement, the design approach is similar for both techniques.

The development and rationale of the various design theories for stone columns are outside the scope of this technical summary. Sufficient design information is presented to assess the feasibility of stone columns. For development of the design theories and in-depth design criteria, FHWA (1983). The publication *The Design of Vibro Replacement* by Priebe (1995) updates earlier work and was a popular and widely used design method. However, for the most current guidance on the design of stone columns, the reader is referred to http://www.GeoTechTools.org.

In practice, the design of stone columns is to a large extent semi-empirical. Specific state-of-the-practice design recommendations are given for bearing resistance, settlement, and stability analyses. These design recommendations give a rational basis upon which to evaluate stone columns. Theoretical results, of course, should always be supplemented by past experience and sound engineering judgment.

The present methods used for analysis and design range from experience based semi-empirical methods to finite element analyses. These methods have been typically indexed to full-scale field tests, laboratory and analytical models to study and predict the load carrying capacity, settlement behavior, shear resistance, and mode of failure of the soil stone-column system.

Weak soils reinforced with stone columns act as a composite medium, exhibiting increased stiffness with reduced spacing, increased column cross-sectional area, and angle of friction for the imported stone. The columns are stiffer than the in situ soils they replace or displace, and rely on the lateral support of the adjacent soil to function properly. Consequently, the columns must have adequate lateral support to preclude a bulging failure and terminate typically in a denser stratum to preclude a bearing resistance failure.

Since the stone column is more rigid than the surrounding soil, it settles less than the adjoining soil under load. Therefore, it carries, by arching, a larger portion of the imposed load. As further consolidation of the in situ soil occurs, additional load transfer to the stone column occurs until an equilibrium condition is reached. This transfer of load to the stiffer, less compressible column, results in decreased settlement for the entire stone-column foundation.

4.1.2 Design Procedure

Stone columns are typically selected to increase bearing resistance, reduce settlement, accelerate consolidation time rate, increase shear strength, reduce liquefaction potential, or provide any combination of the above.

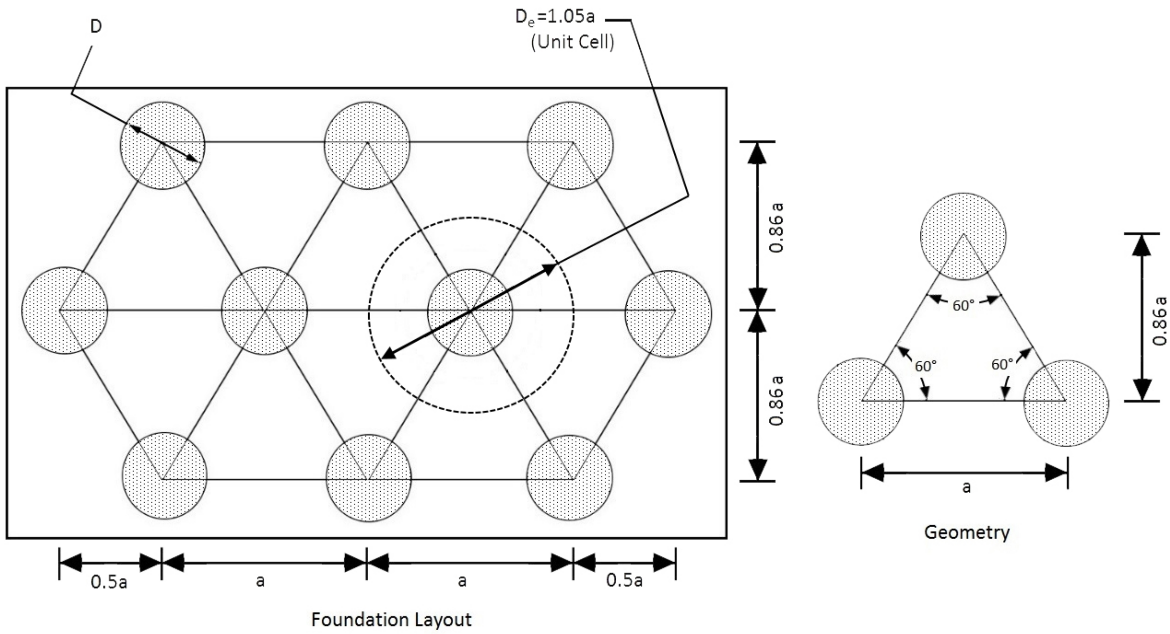
Preliminary design methods and assumptions to achieve the desired end result are outlined in this section.

The generalized design process for embankment support is as follows:

1. Perform embankment design without stone columns to determine the overall settlement and global stability to determine if stone columns or another form of ground modification are required. If yes proceed to step 2.
2. Assume an area replacement ratio and column diameter.
3. Determine the spacing based on the assumed area replacement ratio and column diameter.
4. Check the load bearing resistance of the stone column to see if it meets the project requirements. If not revise the column diameter and re-check.
5. Determine the total settlement of the embankment supported on the stone columns.
6. Check the time rate of settlement. If the time for settlement is too large consider changing the column spacing.
7. Check global stability.

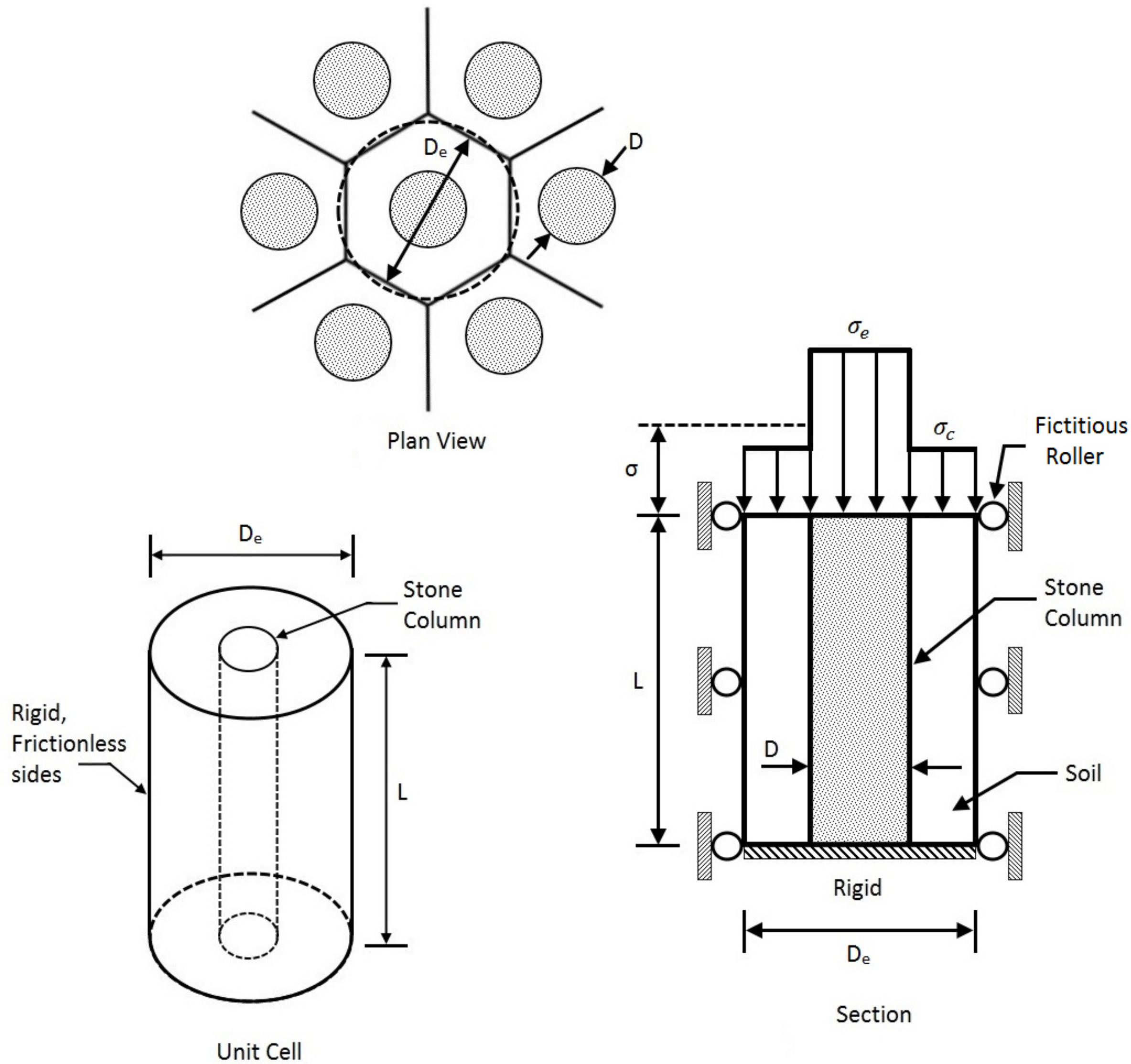
4.1.2.1 Unit Cell Concept

For purposes of settlement and stability analyses, it is convenient to associate the tributary area of soil surrounding each stone column with the column, as illustrated in Figures 5-16 and 5-17.



FHWA 1983

Figure 5-16. Equilateral triangular pattern of stone columns.



FHWA 1983

Figure 5-17. Unit cell idealization.

Although the tributary area forms a regular hexagon about the stone column, it can be closely approximated as an equivalent circle having the same total area. The resulting equivalent cylinder of material having a diameter D_e enclosing the tributary soil and one stone column is known as the *unit cell*. The stone column is concentric to the exterior boundary of the unit cell.

4.1.2.2 Area Replacement Ratio

The volume of soil replaced by stone columns has an important effect upon the performance of the improved ground. To quantify the amount of soil replacement, the *Area Replacement*

Ratio, α_{sc}, is defined as the fraction of soil tributary to the stone column replaced by the stone:

$$\alpha_{sc} = {A_{sc}}/{A} \qquad \text{[Eq. 5-1]}$$

where "A_{SC}" is the area of the stone column after compaction and "A" is the total area within the unit cell. Typical ratios used are in the range of 0.10 to 0.30. The literature also describes the ratio a_{sc}, the *area improvement ratio*, which is the inverse of an area replacement ratio.

4.1.2.3 Spacing and Diameter

Stone column diameters vary between 1.5 and 4 feet, but are typically in the range of 3.0 to 3.6 feet for the dry method, and somewhat larger for the wet method.

Triangular, square, or rectangular grid patterns are used, generally with center-to-center column spacing of 5 to 12 feet. For footing support, they are installed in rows or clusters. For both footing and wide area support, they should extend beyond the loaded area.

4.1.2.4 Stress Ratio

The relative stiffness of the stone column to the in situ soil, as well as the diameter and spacing of the columns, determines the sharing of the imposed area vertical load between the column and the in situ soil.

Since the deflection in the two materials is approximately the same, equilibrium considerations indicate the stress in the stiffer stone column must be greater than the stress in the surrounding soil. The assumption of equal deflection is frequently referred to as an equal strain assumption, which both field measurements and finite element analyses have indicated to be valid.

The stress concentration or stress ratio **n**, defined as the stress in stone column divided by the in situ soil stress, is dependent upon a number of variables, including the relative stiffness between the two materials, length of the stone column, area ratio and the characteristics of the granular blanket placed over the stone column. Measured values of stress ratio have generally been found to be between 2.0 and 5.0, and theory indicates this concentration factor should increase with time. Since secondary settlement in reinforced cohesive soils is greater than in the stone column, the long-term stress in the stone column could be larger than at the end of primary settlement.

For preliminary design, the determination of a design stress ratio is the key element in stone column design; and, unfortunately, it is based largely on experience, although theoretical solutions are available.

A high stress ratio (3 to 4) may be warranted if the in situ soil is very weak and the column spacing very tight. For stronger in situ soils and large column spacings, lower bound stress ratios (2 to 2.5) are indicated. For preliminary design, a ratio of 2.5 is often conservatively used for stability and bearing resistance calculations.

Once a stress ratio has been assumed or determined, the stress on the stone column, σ_{sc}, and on the surrounding soil, σ_{soil}, can be calculated for each replacement ratio, α_{sc}, and any average stress condition, q, that would exist over the unit cell as follows:

$$n = \frac{\sigma_{sc}}{\sigma_{soil}} \quad \text{[Eq. 5-2]}$$

For equilibrium of vertical forces for a given a_{sc}

$$q = \sigma_{sc}\left(\alpha_{sc}\right) + \sigma_{soil}\left(1 - \alpha_{sc}\right) \quad \text{[Eq. 5-3]}$$

For a given stress concentration ratio, the stress on the unimproved soil is:

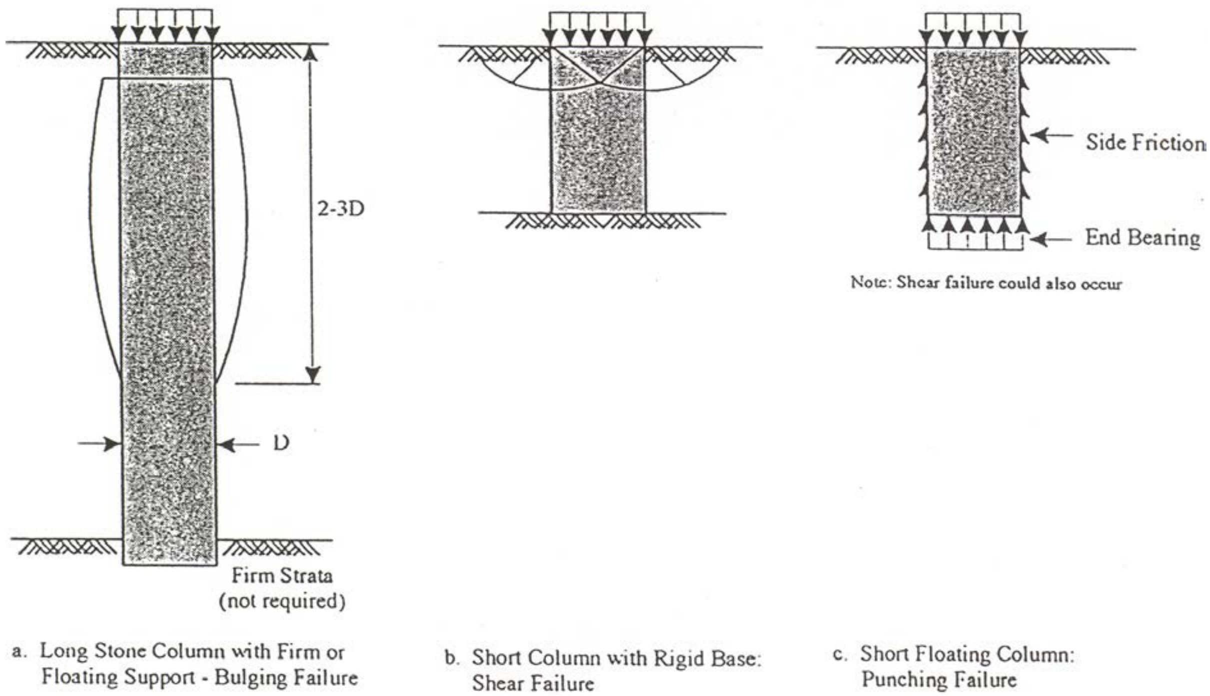
$$\sigma_{soil} = \frac{q}{\left[1 + (n-1)\alpha_{sc}\right]} \quad \text{[Eq. 5-4]}$$

and on the stone column:

$$\sigma_{sc} = \frac{nq}{\left[1 + (n-1)\alpha_{sc}\right]} \quad \text{[Eq. 5-5]}$$

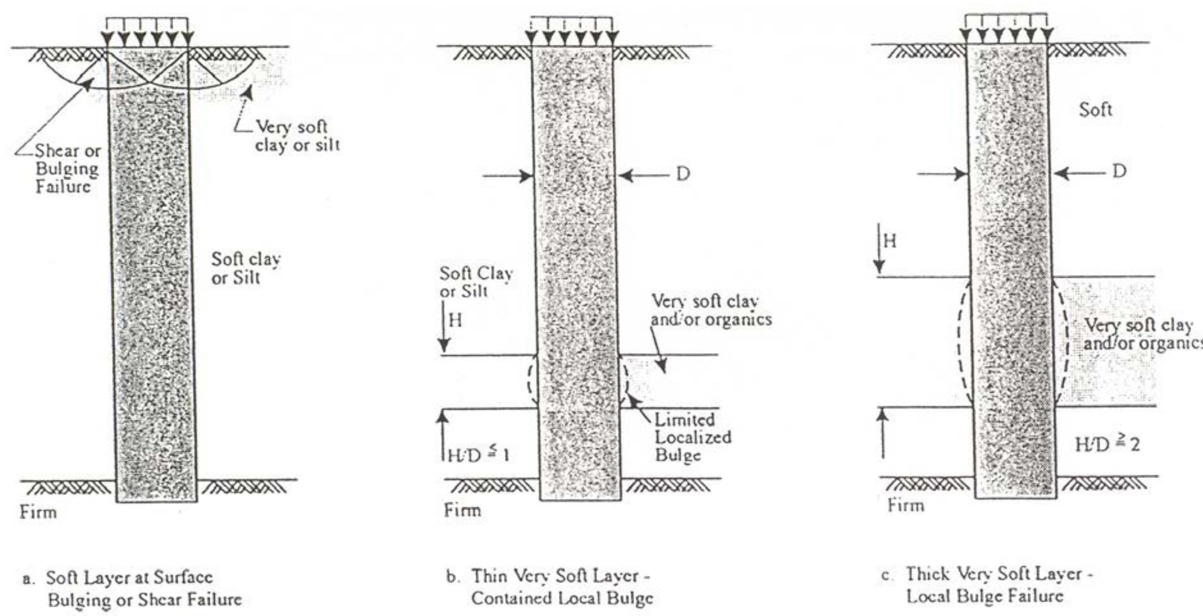
4.1.2.5 Stone Column Vertical Load Capacity

In determining the ultimate load capacity of a stone column or a stone column group, the possible modes of failure to be considered are illustrated in Figures 5-18 to 5-20.



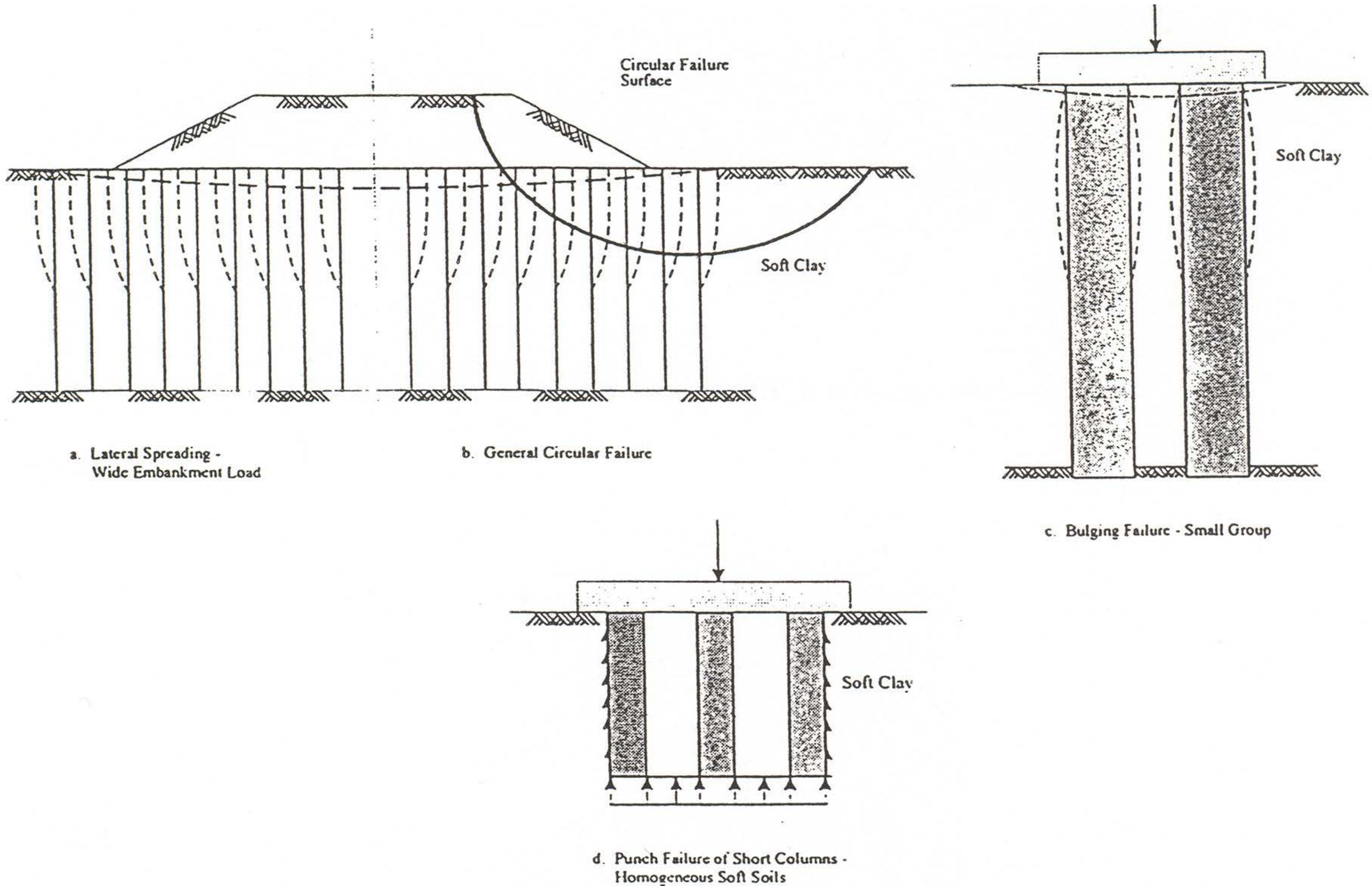
FHWA 1983

Figure 5-18. Failure modes of a single stone column in a homogenous soft layer.



FHWA 1983

Figure 5-19. Failure modes of a single stone column in a non-homogeneous cohesive soil.



FHWA 1983

Figure 5-20. Failure modes of stone column groups.

Caution should be given to avoiding local bulging failures due to very weak or organic layers of limited thickness (Figure 5-19). Bulging would have an effect on the time rate and magnitude of settlement, and may be of concern with respect to stability and stone column shear strength. Use of a bulging analysis for a single column to predict group behavior gives an approximate conservative solution.

The rational prediction of the bearing resistance of stone column groups loaded by either a rigid foundation or a flexible load, such as an embankment, is still in the development stage. As a result, past experience and engineering judgment should be used in addition to theory when selecting a design stone column load.

Frequently, the ultimate capacity of a stone column group is predicted by multiplying the single column capacity by the number of columns in the group. Small-scale model studies using a rigid footing indicate this approach is probably slightly conservative for soft cohesive soils. The bearing resistance of an isolated stone column or a stone column located within a group can be expressed in terms of nominal bearing resistance of the stone column:

$$q_n = c\, N_c \qquad \text{[Eq. 5-6]}$$

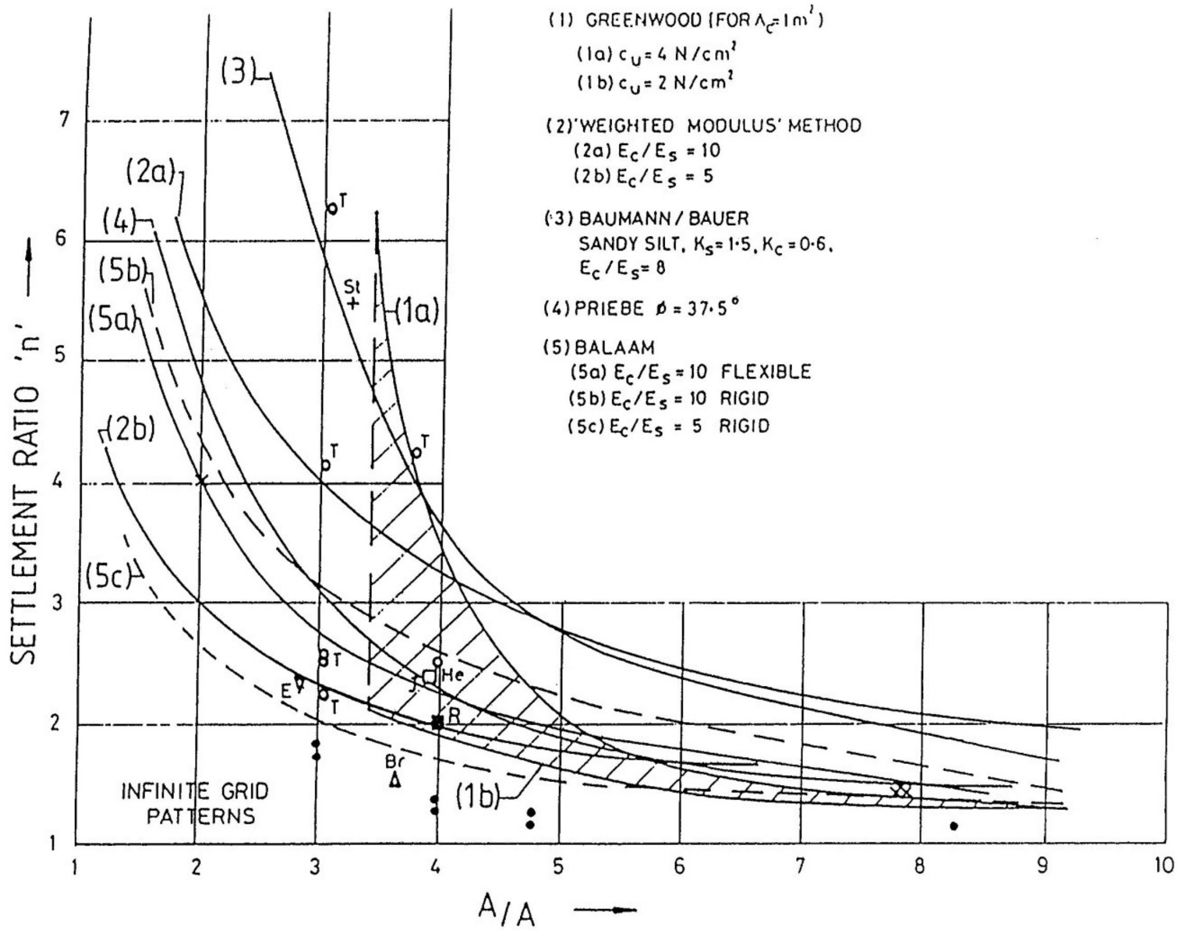
where q_n, c, and N_c are the nominal bearing resistance of the stone column can carry, the undrained shear strength of the surrounding cohesive soil, and the bearing capacity factor for the stone column, respectively. Bearing capacity factors between 18 and 22 have been found to provide good estimates.

Cavity expansion theory indicates that the ultimate capacity and, hence, N_c is dependent upon the compressibility of the soil surrounding the stone column. Hence, soils with organics or other soft clays would be expected to have a smaller value of N_c compared to stiffer soils. For soils having a reasonably high initial stiffness, an N_c of 22 is recommended; for soils having low stiffness, an N_c of 18 is recommended. Low stiffness soils would include peats, organic cohesive soils, and very soft clays with plasticity indices greater than 30. High stiffness soils would include inorganic soft-to-stiff clays and silts. The recommended values of N_c are based on a back-analysis of field test results. In this analysis, the strengths of both the soil and stone column were included. A resistance factor of 3 is recommended for design if using Equation 5-6.

Typically, single column design loads of 40 to 60 kips can be used in soft to medium stiff clays.

4.1.2.6 Settlement

Reduction of settlement is one of the improvement benefits achieved by the use of stone columns. The reduction of settlement has been estimated by both pseudo-elastic and elasto-plastic methods, considering both isolated and wide spread loading using a unit cell concept. The predicted improvement, often expressed as the settlement ratio "n", defined as the ratio of settlement without stone columns to that with stone columns, is typically related to the area replacement (α_{sc}) or area improvement ($1/\alpha_{sc}$) ratio. The settlement of the non-improved zone is determined by conventional settlement analyses. Improvement predictions based on some theoretical analytical methods, as well as results from field measurements, are shown in Figure 5-21 (Greenwood and Kirsch 1984).



Wallays et al. 1983

Figure 5-21. Comparison of elastic theories and field observations.

Han (2015) presents three methods for calculating the settlement of granular column reinforced foundations. The three are stress reduction method, improvement factor method, and elastic-plastic method.

It should be noted in Figure 5-21 that the settlement ratio “n” was determined analytically by various researchers as a function of the ratio of the Modulus of the stone column (E_{sc}) to the in situ soil modulus (E_{soil}), or a measure of the strength of the stone column (N) to the shear strength of the in situ soil (c_u).

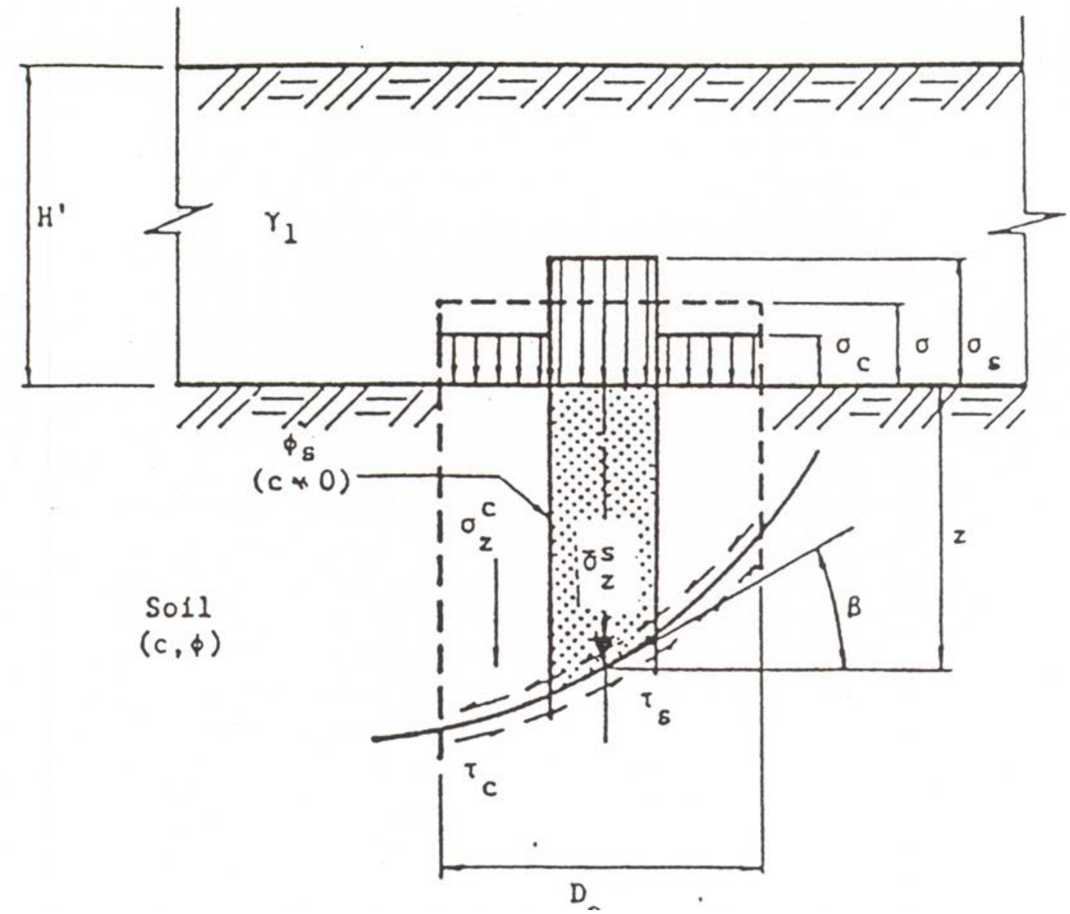
For preliminary estimates, the Priebe curve may be used to evaluate the upper bound effectiveness and cost at various spacings. It should be further noted that the Equilibrium Method outlined in FHWA (1983) *Design and Construction of Stone Columns* is roughly equivalent to the Balaam relationships shown in Figure 5-21 and represents an average or lower bound estimate suitable for preliminary analyses.

4.1.2.7 Rate of Settlement

Stone columns substantially alter the time-rate of settlement as radial drainage governs. Therefore, time-rate of settlement computations are identical to the computations performed for vertical sand drains and prefabricated vertical drains (see Chapter 2 –Prefabricated Vertical Drains). The effect of disturbance or smear during installation, which reduces radial flow, can be roughly accounted for by reducing the diameter of the column by 50 to 80 percent of its design diameter. A larger disturbance or smear zone should be anticipated with the dry-displacement construction method and for all installations in sensitive clays.

4.1.2.8 Shear Strength Increase

For slope stability analyses, an average shear strength of the soil/stone column composite material has been used in the past to estimate the stability of the embankment. How this approach is used is illustrated in Figure 5-22.



FHWA 1983

Figure 5-22. Notation used in average stress method stability analysis.

However, recent research has shown that the average strength approach may overestimate the factor of safety by 10% for an undrained condition (Zhang et al. 2014; Abusharar and Han 2011).

The composite strength is a function of the undrained shear strength of the in situ soil, the frictional resistance of the column, the area replacement ratio, the stress ratio, and the loading condition. For significant improvement to occur, a relatively close spacing and a substantial overburden pressure is necessary to mobilize the frictional strength of the column.

The average strength, τ, and average unit weight, $\bar{\gamma}$, parameters can be determined as follows:

$$\tau = (1-\alpha_{sc})c_u + \alpha_{sc}\,\sigma_v \tan\phi_{sc} \qquad \text{[Eq. 5-7]}$$

$$\bar{\gamma} = \gamma_{sc}\,\alpha_{sc} + \gamma_{soil}\,(1-\alpha_{sc}) \qquad \text{[Eq. 5-8]}$$

where,

τ = average weighted shear strength

c_u = undrained shear strength of in situ soil

$\bar{\gamma}$ = average unit weight

γ_{soil}, γ_{sc} = unit weight of soil and stone column

ϕ_{sc} = angle of friction for stone column

σ_v = stress due to embankment loading

α_{sc} = area replacement ratio

For design, the angle of internal friction ϕ_s of the stone column typically used varies from 40 to 45 degrees. The lower angles should be considered for gravel mixtures, and the higher angles for angular crushed stone mixtures.

Note that in landslide remediation projects, the stress ratio is 1, and consequently the strength parameters are essentially a weighted average.

Stability analyses may be performed using a total stress approach by assigning $\phi = 0$ for end-of-construction conditions, or, using an effective stress approach, by assigning $c = 0$ for long-term conditions. A target factor of safety of 1.2 to 1.3 is considerate adequate.

Over the last decade slope stability programs have advanced to the point where it is now easy to model discrete stone columns and not use the average shear strength method described above. For a complete description of the stability analysis methods using discrete stone columns in the model, see http://www.GeoTechTools.org.

4.1.3 Seismic Design

In the United States, there has been an effort to evaluate the liquefaction potential of soils from in situ density data and to modify and improve the properties of these soils. *"Quantitative Evaluation of Stone Column Techniques for Earthquake Liquefaction Mitigation"* (Baez and Martin, 1992), *Soil Improvement for Earthquake Hazard Mitigation,* (Hryciw Editor 1995), *Advances in the Design of Vibro Systems for Improvement of Liquefaction Resistance* (Baez 1993) and *Review of Verification and Validation of Ground Improvement Techniques for Mitigation of Liquefaction* (Woeste et al. 2016) provide recommendations on how to quantify the benefits of ground modification using stone columns and how to evaluate the actual safety factor against seismic liquefaction. The benefits of stone columns with respect to liquefaction mitigation are that the soil around the column is densified, the drainage of excess pore water is facilitated, and the stiffer (i.e., the stone column is stiffer than the surrounding soil) stone column accepts higher seismic stress than the surrounding soil. The approach presented below is a simplified procedure that only considers the benefit of soil densification. This approach is appropriate for preliminary designs and more rigorous analysis may be warranted for final design.

4.1.3.1 Soil Density

It is well understood that under cyclic loading, pore pressure generation in a dense soil occurs more slowly than in loose sand. Therefore, liquefaction potential can be reduced by increasing soil density. For loose sands, once the state of initial liquefaction is reached, large ground deformations may occur due to their lower initial strength. In dense sands, when peak pore pressures become equal to the initial confining pressure, the larger shear strains mobilize significant dilation of the sand structure, thereby maintaining significant residual stiffness and strength.

Densification has been used frequently for reducing the potential for liquefaction. Seed et al. (1985) developed empirical liquefaction curves which correlate cyclic stress ratio to corrected penetration resistance. The cyclic stress ratio (CSR_{EQ}) is determined as follows:

$$CSR_{EQ} = 0.65 \left(a_{\max} / g \right) r_d \left(\sigma_v / \sigma_v' \right) \qquad \text{[Eq. 5-9]}$$

where,

a_{max}	=	maximum ground acceleration
σ_v	=	total vertical stress at any depth z
σ_v'	=	effective vertical stress at any depth z
r_d	=	stress reduction factor

For preliminary designs the stress reduction factor may be estimated as a function of depth (z) based on the following (note that the following equations were developed for metric units):

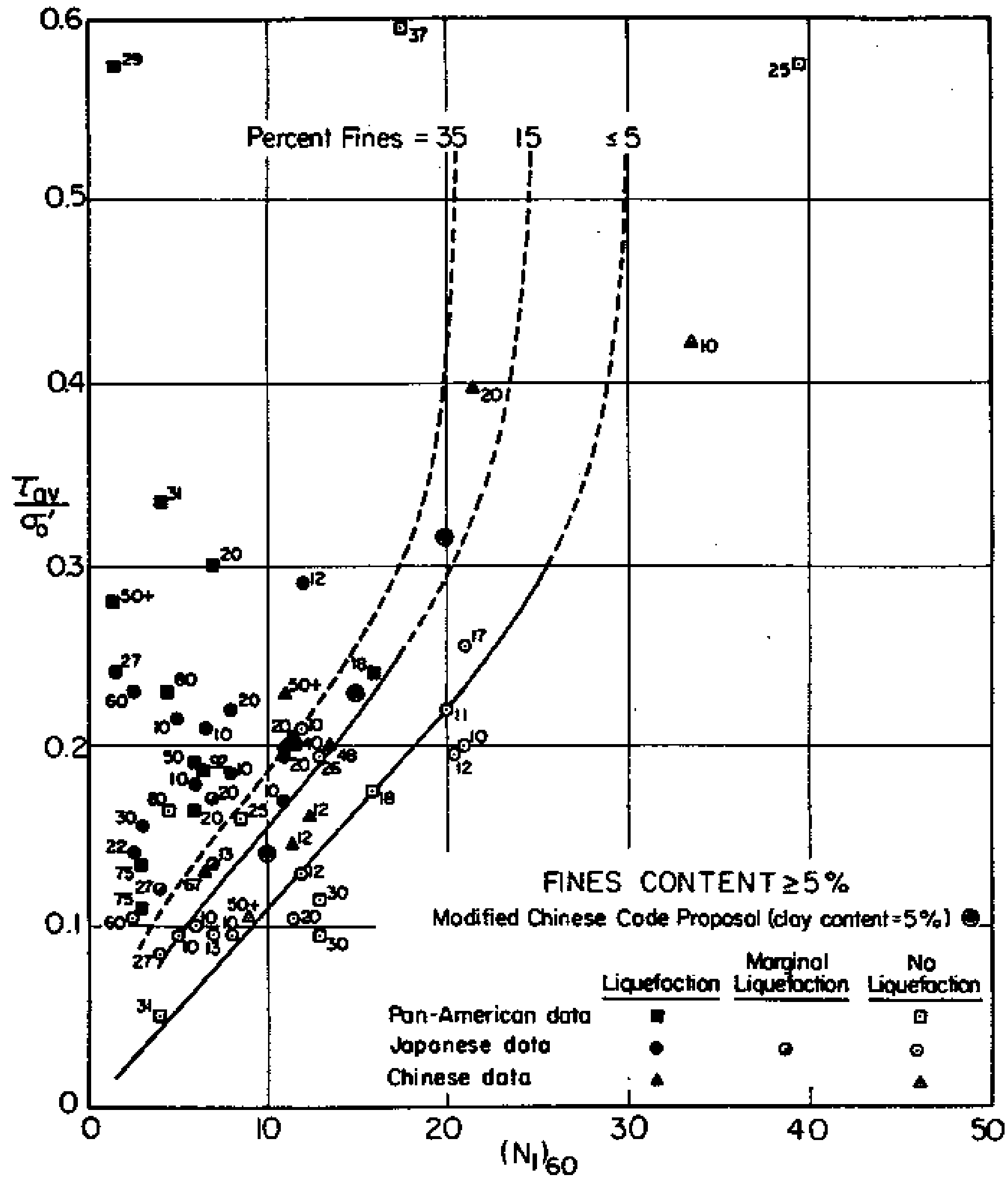
r_d = 1.0 - 0.00765z for $z \leq 30$ ft.

r_d = 1.174 - 0.0267z for 30 ft. < z ≤ 75 ft.

r_d = 0.744 - 0.008z for 75 ft. < z ≤ 100 ft.

r_d = 0.5 for z < 100 ft.

The relationship between stress ratio causing liquefaction and corrected SPT "$(N_1)_{60}$" values for silty sands for a magnitude 7.5 earthquake is shown in Figure 5-23. This figure may be used to estimate the required improvement in soil density to prevent liquefaction.



Seed et al. 1985.

Figure 5-23. Relationship between stress ratio causing liquefaction and $(N_1)_{60}$ values for silty sands for 7.5 magnitude earthquakes.

4.1.3.2 Spacing

The spacing of the stone columns may be determined using Figure 5-23 to determine the corrected SPT "N" value. Use Table 4-8 from the Chapter 4 Deep Compaction to correlate SPT "N" values to relative density. Then use Figure 4-34 (from Chapter 4 Deep Compaction) to estimate the required stone column spacing to improve the soil to the required penetration resistance at the mid-point between columns.

4.1.3.3 Permeability

In order to avoid significant generation of pore water pressures within the stone column, it is recommended that the permeability of the stone column be at least two orders of magnitude larger than the treated soil. This recommendation can be achieved by selection of the gradation for the stone column, with due regard to piping considerations outlined below.

4.1.3.4 Piping Prevention

There is a likelihood that hydraulic gradients may exceed critical gradients (greater than one). This situation may initiate a movement of fines from the natural soil into the large, open pore structure of the stone column during seismic loading, leading to the development of cavities within the soil structure and potentially undesirable volume change. In reality, due to the short duration of the strong motion, it is unlikely that much soil material could be carried into the stone column.

Based on experimental data, the following relationship is recommended for piping prevention under any loading condition based on the grain size distribution of the stone column and the surrounding soil. Adherence to these criteria will ensure maximum permeability and prevent piping of the soil:

$$20\,D_{S15} < D_{G15} < 9\,D_{S85} \qquad \text{[Eq. 5-10]}$$

where D_{S15} is the diameter of soil particle passing 15 percent, D_{G15} is the diameter of gravel (stone) passing 15 percent, and D_{S85} is the diameter of soil particle passing 85 percent in a grain size analysis test.

4.2 Rammed Aggregate Piers

4.2.1 Design Considerations

The design concept used for rammed aggregate piers is almost identical to that used for stone columns.

For area wide ground improvement applications, the design method is identical to that previously detailed for stone columns. The stone within the rammed aggregate piers having been compacted by impact ramming typically exhibits a somewhat higher effective friction angle, in the range of 45 to 50 degrees, and potentially higher stiffness (modulus). Consequently, the ratio of the stiffness (modulus) of the rammed aggregate piers to the stiffness of the in situ soil should be somewhat higher than for stone columns, resulting in a higher design stress ratio than previously identified for stone columns. Although the Geopier design manual suggests stress ratios of 20 or higher, a stress ratio of 5 to 10 for area ground improvement applications under flexible embankment loading appears warranted until considerably more field data in support of a higher ratio is developed.

For structure foundation support under rigid footings a somewhat higher stress ratio (10) may be considered, with anticipated settlements and pier capacity conventionally computed, based on the loads on each element. The load on the rammed aggregate pier and on the in situ soil is based on the chosen stress and replacement ratios. The design area replacement ratio is determined after evaluating settlement of the unimproved soil. A minimum replacement ratio of 0.33 is generally recommended, as noted in the HITEC Evaluation Report (Collin 2007).

4.2.2 Design Procedures

Rammed aggregate piers are typically selected to increase bearing resistance, reduce settlement, increase shear strength, or provide any combination of the above.

Preliminary design methods and assumptions to achieve the desired end result are outlined in this section.

The generalized design process for an embankment support is as follows:

1. Perform embankment design without rammed aggregate piers to determine the overall settlement and global stability to determine if rammed aggregate piers or another form of ground improvement are required. If so proceed to step 2.
2. Assume an area replacement ratio and column diameter.
3. Determine the spacing based on the assumed area replacement ratio and column diameter.
4. Check the load bearing resistance of the rammed aggregate pier to see if it meets the project requirements. If not revise the column diameter and re-check.
5. Determine the total settlement of the embankment supported on rammed aggregate piers.

6. Check the time rate of settlement. If the time for settlement is too large consider changing the column spacing.
7. Check global stability.

The design process for rammed aggregate piers is similar in many respects to stone columns. However, the major difference in design of the two systems is with respect to settlement analysis that is presented in the following sections.

4.2.3 Settlement Analysis

Rammed aggregate pier settlement control design methodology is based on a two-layer settlement approach as described by Lawton et al. (1994), Fox and Cowell (1998), and Wissmann et al. (2002). The installation of rammed aggregate piers within the aggregate column-reinforced zone, referred to as the upper zone, creates a stiffened, engineered zone with reduced compressibility that reduces settlement of embankments and transportation-related structures. The settlement below the rammed aggregate pier-reinforced zone, referred to as the lower-zone, is evaluated using conventional geotechnical analysis approaches. The total settlement (s_{tot}) of the transportation structures is evaluated as the sum of the upper zone settlement (s_{uz}) and the lower zone settlement (s_{lz}):

$$s_{tot} = s_{uz} + s_{lz} \qquad \text{[Eq. 5-11]}$$

4.2.3.1 Settlement in the Rammed Aggregate Pier-Reinforced Zone

Settlement in the rammed aggregate pier-reinforced zone (upper zone) is estimated by first calculating the top-of-pier stress (q_g) using the following equation:

$$q_g = q\left[\frac{n_s}{n_s R_a - R_a + 1}\right] \qquad \text{[Eq. 5-12]}$$

where,

q	=	average applied bearing pressure
R_a	=	ratio of the cross-sectional area coverage of the rammed aggregate piers to the matrix soil
n_s	=	stress concentration ratio between the rammed aggregate piers and the matrix soil

Research has shown that stress concentration ratios range from 4 to 45 for rigid footings. Because embankments and most MSE walls are not rigid structures, stress concentration ratios may be lower than those observed for rigid footings and should be selected with care. In addition, the stress concentration ratio is related to the stiffness of the matrix soil with larger ratios resulting at softer soil sites. Suggested stress concentration ratios ranging from 5 to 10 may be used for settlement control of embankments.

The settlement of the rammed aggregate pier-reinforced zone is estimated as the top-of-rammed aggregate pier stress, q_g, divided by the rammed aggregate pier stiffness modulus, k_g,:

$$s_{uz} = \frac{q_g}{k_g} \qquad \text{[Eq. 5-13]}$$

Design rammed aggregate pier stiffness modulus values range from 75 pci to 360 pci for support of rigid footings. Conservative stiffness modulus values should be used for support of embankments and transportation-related structures (Collin 2007).

4.2.3.2 Settlement below the Rammed Aggregate Pier Reinforced Zone

Settlement below the rammed aggregate pier-reinforced zone is evaluated using conventional geotechnical approaches, consisting of either elastic settlement analyses or consolidation analyses using equation 5-14 for cohesionless or overconsolidated cohesive soils and equation 5-15 for normally-consolidated cohesive soils

$$s_{lz} = \frac{\Delta q\, H}{E} \qquad \text{[Eq. 5-14]}$$

$$s_{lz} = c_c \left(\frac{1}{1+e_o} \right) H \log\left(\frac{p_o + \Delta q}{p_o} \right) \qquad \text{[Eq. 5-15]}$$

where H is the thickness of the lower zone, E is the matrix soil elastic modulus within the lower zone, c_c is the matrix soil coefficient of compressibility, e_o is the matrix soil void ratio, p_o is the vertical effective stress at the mid-point of the compressible layer, and Δq is the average bearing pressure applied by the embankment. The average applied bearing pressure is the product of the applied pressure and the stress influence factor, I_σ. The stress influence factor can be determined using either Boussinesq or Westergaard's method. Rammed aggregate pier reinforced soil is typically considered a layered soil and therefore Westergaard's method is typically used. However, for embankments, the stress influence factor within the lower zone is typically assumed to be 1.0 because of the large lateral extent of embankment fills.

Typically, elastic modulus settlement approaches are used to estimate settlement in granular soils and heavily over-consolidated cohesive soils. Matrix soil equivalent elastic modulus values may be estimated using published correlations from SPT N-values, undrained shear strengths, CPT tip resistances, or other in situ tests. Consolidation settlement approaches are used to evaluate settlement in normally-consolidated or lightly over-consolidated cohesive soils.

4.3 Design Examples

4.3.1 Rammed Aggregate Piers

The following design example is provided to demonstrate the method to determine settlement of an embankment supported on rammed aggregate piers.

4.3.1.1 Problem

A new embankment is to be constructed over a soft clay layer that is underlain by rock. The geometry of the embankment and soil stratigraphy are shown in Figure 5-24.

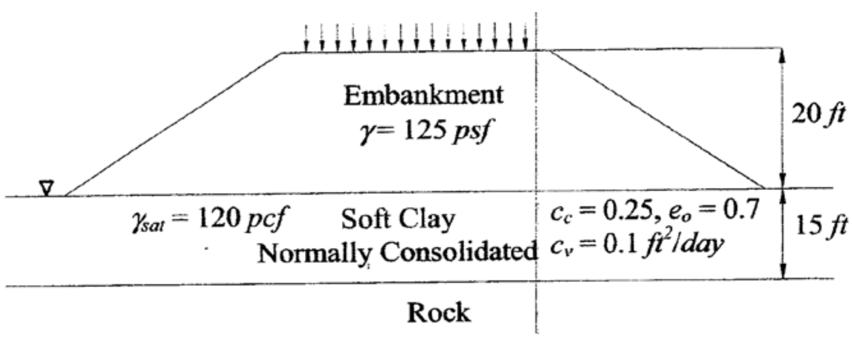


Figure 5-24. Example problem 1 geometry.

Determine the total settlement that will occur after the embankment is constructed. The spacing of the columns is 5 feet and the diameter of the columns is 2.75 feet.

4.3.1.2 Total Settlement Magnitude without Ground Improvement

$$P_o = z(\gamma_{sat} - \gamma_w) = 7.5\ ft(120\ pcf - 62.4\ pcf) = 432\ psf$$

$$q = \gamma H = 125\ pcf\ (20\ ft) = 2{,}500\ psf$$

$$S = c_c \frac{1}{(1+e_o)} H \left[\log\left(\frac{P_o + \Delta q}{P_o} \right) \right] = 0.25 \left(\frac{1}{1+0.7} \right) (15) \log\left(\frac{432+2500}{432} \right) = 1.83\ ft = 22\ inches$$

The total expected settlement of the embankment without ground improvement is 22 inches. The proposed aggregate column layout for the embankment is shown in Figure 5-25.

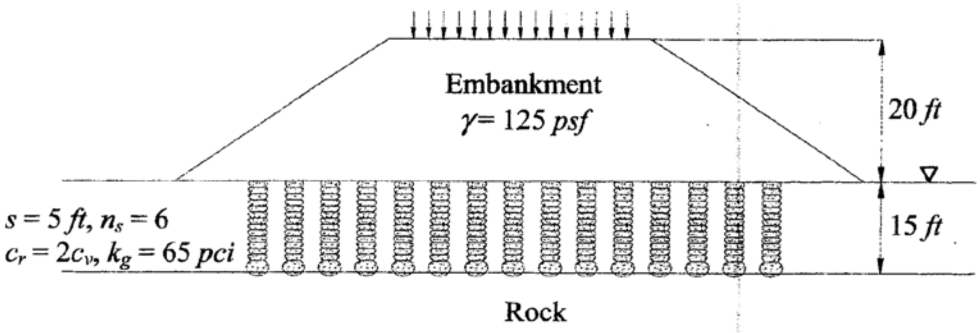


Figure 5-25. Aggregate column ground improvement layout.

Determine the anticipated amount of settlement with the rammed aggregate piers.

4.3.1.3 Settlement Magnitude with Rammed Aggregate Piers

$$R_a = \frac{A_g}{a} = \frac{5.94\ ft}{(5.25\ ft)^2} = 0.27$$

$$d_e = 1.05 s = 1.05(5) = 5.25$$

$$q = \gamma H = 125\ pcf\ (20\ ft) = 2{,}500\ psf$$

$$q_g = q \left[\frac{n_s}{n_s R_a - R_a + 1} \right] = 2{,}500\ psf \left[\frac{6}{6(0.27) - 0.27 + 1} \right] = 6{,}383\ psf$$

$$s_{uz} = \frac{q_g}{k_g} = \frac{6{,}383\ psf}{(65\ pci)\left(144\ in^2/ft^2 \right)} = 0.68\ inches$$

4.3.2 Stone Columns

4.3.2.1 Problem

A new embankment is to be constructed over a soft clay layer that is underlain by dense sand with and an average $N1_{60} = 48$. The geometry of the embankment and soil stratigraphy are shown in Figure 5-26.

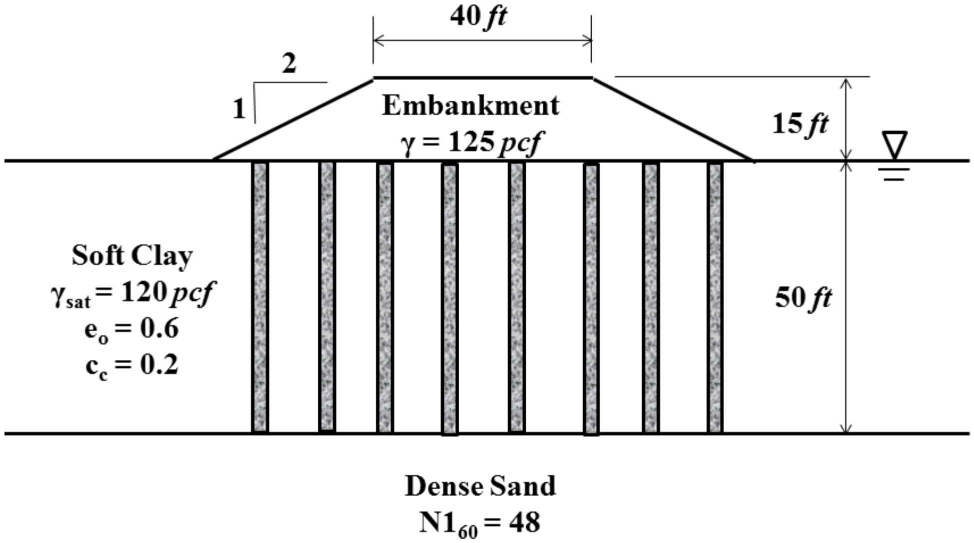


Figure 5-26. Example problem 2 geometry and soils.

Determine the total settlement that will occur after the embankment is constructed. The spacing of the columns is 5.7 feet and the diameter of the columns is 3.0 feet. Assume that no settlement on the dense sand will occur.

4.3.2.2 Total Settlement Magnitude without Ground Improvement

$$P_o = z(\gamma_{sat} - \gamma_w) = 25\ ft(120\ pcf - 62.4\ pcf) = 1{,}440\ psf$$

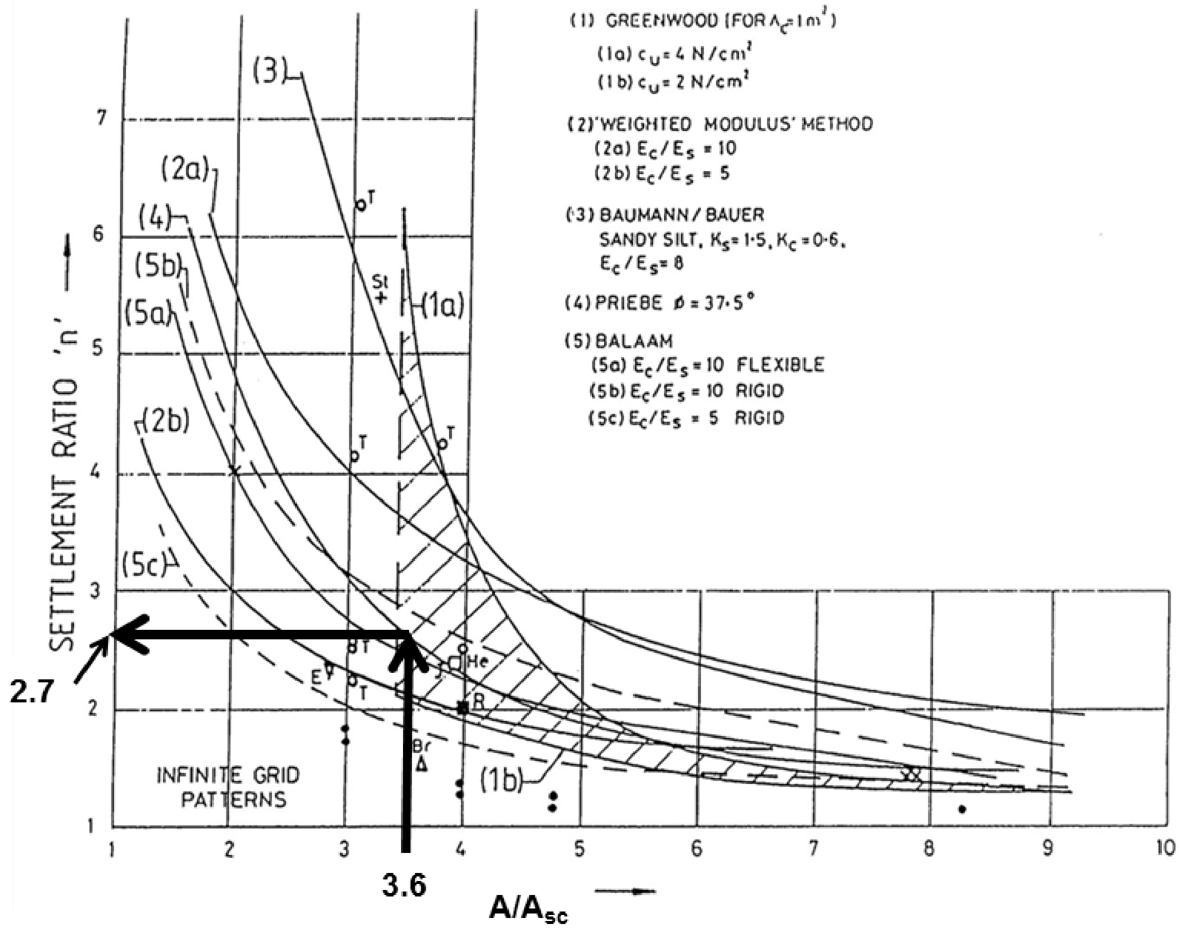
$$q = \gamma H = 125\ pcf\ (15\ ft) = 1{,}875\ psf$$

$$S = c_c \frac{1}{(1+e_o)} H \left[\log\left(\frac{P_o + \Delta q}{P_o}\right)\right] = 0.2\left(\frac{1}{1+0.6}\right)(50)\log\left(\frac{1{,}440+1{,}875}{1{,}440}\right) = 2.26\ ft = 27\, inches$$

4.3.2.3 Settlement Magnitude with Stone Columns

$$\frac{A}{A_{sc}} = \frac{(5.7)^2}{(3.0)^2} = 3.6$$

Using the Priebe curve from Figure 5-27, determine the settlement ratio.



After Wallays et al. 1983

Figure 5-27. Example problem 2 settlement ratio determination.

The settlement ratio is 2.7. Therefore, the settlement of the embankment using stone columns as ground improvement is 10 inches.

4.4 Design Verification

As an important adjunct to design, a field verification program of load tests and in situ testing must be developed and implemented through appropriate construction specification requirements. A program should be specified, regardless of the contracting method.

A combination of load tests on aggregate columns constructed before, during, and after production should be specified to verify the design assumptions and the performance specification. There are three types of load tests: (1) short-term tests, which are used to evaluate ultimate stone column bearing resistance, (2) long-term tests, which are used to measure the consolidation settlement characteristics; and (3) horizontal or composite shear tests, which are used to evaluate the composite aggregate-soil shear strength for use in stability analyses. The most common of these tests is the short-term load test on a single column.

The short-term load tests, similar to pile load tests, should be performed after all excess pore pressures induced during construction have been dissipated. The load increment should closely correspond to the actual loading. For example, if the actual foundation load will be applied very slowly, a load increment of approximately 10 percent of the ultimate should be used. A rapid loading may result in immediate settlement, as well as consolidation settlement. If the actual load will be applied rapidly, a load increment of 20 to 25 percent of ultimate should be used. The tests are generally performed to 150% of the design load, and the measured settlement is compared to project settlement tolerance. For example, a final acceptance criterion of 1 inch of settlement at 150 to 200 percent of the design load appears to be a reasonable criterion for columns supporting a structure.

The long-term settlement of the stone column foundation is usually estimated from the results of short-term load tests on single stone columns. Mitchell (1981) reported that the foundation settlement due to a uniform loading of a large area was 5 to 10 times greater than the settlement measured in a short-term load test on a single column. However, there is very little field data available to confirm this behavior. Therefore, it is recommended that long-term load tests on a group of columns be conducted in conjunction with short-term load tests to develop an estimate of the settlement of the stone column foundation. The long-term load tests should be conducted on a minimum of three to four stone columns located within a group of 9 to 12 columns having the proposed spacing and pattern. The load should be applied over the tributary area of the columns and left in place until the cohesive soil reaches a primary degree of consolidation of 90-95 percent. The applied load could consist of column backfill material, native material, and/or the dead weight from the short-term load tests. The results of these tests will provide valuable information for estimating the ultimate settlement of the stone column foundation.

During the production phase of construction, a few short-term load tests can be performed for quality control purposes. These tests are referred to as proof tests and are used to verify quality control during production. The load applied in the proof test is usually 150 to 200 percent of the allowable/design load.

In situ testing to evaluate the effect of the stone column construction on the native cohesive soil can be also specified. However, the specified test method should be selected on the basis of its ability to measure changes in lateral pressure in cohesive soils. The cone penetrometer (CPT), the flat plate dilatometer (DMT), and the pressuremeter (PMT) appear to provide the best means for measuring the change, if any, in lateral stress due to stone column construction. Due to the limited amount of information that will be obtained from CPT, DMT, or PMT testing after column construction, it is recommended that long-term load tests on groups of stone columns be conducted instead of in situ tests. However, extensive in situ testing should be conducted during the initial subsurface investigation to reliably estimate the soil profile and the stone column design parameters.

For rammed aggregate pier construction, a Modulus test and a Bottom Stabilization test have been developed and are used as quality assurance checks. For details, consult the HITEC Evaluation Report (Collin 2007).

5.0 CONSTRUCTION SPECIFICATIONS AND QUALITY ASSURANCE

Like other methods of specialty construction, unless the specifying agency has expertise in the design, construction, and inspection of aggregate columns, it is good practice to specify that the work be accomplished under a performance type specification. If the specifying agency has the necessary experience with the aggregate column technique, a method specification may be utilized.

5.1 Aggregate Column Performance Specification

As part of the development of *GeoTechTools*, an extensive evaluation was made of specifications for aggregate columns. Twenty-one specifications written by state DOTs and other agencies were reviewed and evaluated. Of the assessed specifications, two specifications were only applicable to rammed aggregate piers, 14 specifications were only applicable to stone columns, and five specifications were applicable to both rammed aggregate piers and stone columns. These specifications were used to develop a guide specification entitled *Guide Specification for Aggregate Columns* that is intended to be a complete and fair specification containing commentary and instructions that are easily adaptable by the user for a specific project. This guide specification can be accessed at http://www.GeoTechTools.org under the Aggregate Columns Technology Information page and is applicable to both stone columns and rammed aggregate piers).

An outline of the current *Guide Specification for Aggregate Columns*, illustrating what items should be contained in such a specification, follows.

PART 1 GENERAL

- 1.01 INTRODUCTION
- 1.02 INTENT
- 1.03 STANDARDS AND REFERENCES
- 1.04 DEFINITIONS
- 1.05 SCOPE OF WORK
- 1.06 SUBMITTALS
- 1.07 QUALIFIED CONTRACTORS
- 1.08 QUALITY ASSURANCE

PART 2 EQUIPMENT AND MATERIALS

2.01 EQUIPMENT

2.02 BACKFILL MATERIALS

PART 3 EXECUTION

3.01 SITE INSPECTION

3.02 AGGREGATE COLUMN CONSTRUCTION

3.03 PERFORMANCE CRITERIA

3.04 FIELD QUALITY ASSURANCE

3.05 REJECTION OF AGGREGATE COLUMNS

3.06 EXCAVATION OF COLUMNS TOPS, AND UTILITIES

3.07 SUBGRAD PREPARATION

3.08 RESTRICTIONS

PART 4 PAYMENT

4.01 METHOD OF PAYMENT

5.2 Field Inspection and Improvement Verification

Verification and detailed field inspection of aggregate column construction is a very important, but often neglected, aspect. Thorough field surveillance by both the Engineer and Contractor is essential in the construction of aggregate columns. Furthermore, good communication should be maintained at all times between the inspection personnel, Contractor, Project Engineer and Designer.

5.2.1 Stone Columns

A comprehensive stone column Quality Assurance (QA) assessment program usually consists of several QA methods. Gradation, specific gravity, loose density, and compacted density tests should be run on the stone to be installed, with a frequency of one test for each 5,000 tons of material prior to construction to ensure compliance with specifications. Stone column performance is dependent upon the integrity of the column. It is important that the minimum column diameter and required compacted density of the stone be achieved in order to ensure the desired performance. During construction, stone consumption, in terms of buckets of a known weight or volume, should be monitored as a function of depth. Based on the loose and in-place, compacted density of the stone, it is possible to estimate the column diameter. Barksdale and Bachus (FHWA 1983) provide a method for estimating the in-place density of the stone based on loose and compacted density tests. Measurements should typically be taken at a maximum of 5-foot increments to determine the column's cross-sectional area profile versus depth. Decreased rate of stone consumption may indicate caving of the hole or failure to attain adequate displacement and replacement of the surrounding ground. For any group of 50 consecutively installed stone columns, the average diameter over the total length should not be less than as specified in the contract documents. No stone column should have a diameter less than 90% of the minimum diameter specified in the contract documents. Verticality of the rig should be monitored, and no stone column axis should be inclined from the vertical by more than 2 inches in 10 feet. During construction of the column, each lift should be re-penetrated until the specified amp-meter reading is achieved, thus indicating good input energy from the vibrator probe to the stone. In general, it is recommended that, as a minimum, the vibrator free-standing current reading plus at least 40 additional amps be developed.

For projects requiring the improvement of large areas, it is desirable to subdivide the total area into approval or acceptance zones on the order of 100 feet on a side. Completing the work with timely approval on a zone-by-zone basis means that the contractor may proceed without risk of having to return late in the project to correct deficiencies that developed early in the project.

All construction records should be furnished to the engineer, with the following data to be obtained during column installation:

- Stone column reference number
- Measurement of rig verticality
- Elevation of top and bottom of each stone column
- Number of buckets of stone backfill in each stone column
- Amperage achieved as a function of depth; the date and column identification should be written on each record
- Time to penetrate and time to form each stone column
- Details of obstructions, delays, and any unusual ground conditions
- Digital data log of amperage, depth, and stone consumption

Post-construction QA is dependent on the specific application and the type of ground in which the stone columns are installed. For slope stabilization, structure or embankment support, settlement reduction, liquefaction mitigation, and prevention of lateral spreading applications in silty and clayey sands where densification is required, in situ testing (SPT, CPT, or PMT) should be conducted at central points between the columns. Penetration resistance should be verified against values that were used to determine column spacing. The same test method should be utilized both before and after the stone column installation to verify soil improvement.

Stone column installation is not expected to induce densification of soft, saturated clays. If the columns are to support a structure or embankment in such soils, load tests are sometimes required to determine the short-term capacity and settlement of the column. Short-term load tests should be conducted in accordance with ASTM D1143, *Standard Test Methods for Deep Foundations Under Static Axial Compressive Load*, on individual columns after all pore pressures induced by construction have dissipated. If settlement is a primary concern, longer-term load tests are highly recommended, with settlement readings generally taken over a one-week period. The longer-term load tests should be conducted on a minimum of three to four stone columns located within a group of nine to 12 columns having the proposed spacing and pattern. The load should be applied over the tributary area of the columns and may consist of column backfill material, native material, and/or the dead weight from the short-term load tests. Concrete blocks and reaction pile systems may also be used for load testing of single columns. Surveying methods should be used to ensure proper column spacing and location. No column should be more than 4 inches from the specified center location unless an obstruction is encountered. In case of an obstruction, the Engineer should be notified to determine the maximum allowable offset. Gradation analyses on samples taken from installed columns may be used to confirm that the in situ gradation matches the specifications and that the columns have not been penetrated by excessive amounts of fines from the surrounding ground. Such testing may be appropriate for the owner's information in a method specification, but columns cannot be rejected for failing to meet a post-installation gradation criterion if other provisions of a method specification have been followed.

5.2.2 Rammed Aggregate Columns

A comprehensive rammed aggregate pier QC/QA assessment program usually consists of several QC/QA methods. It is the responsibility of the QC representative to coordinate with the General Contractor on footing layout and pier elevations, observe installation procedures, ensure the aggregate moisture content is within acceptable limits, perform tests on production piers, and implement corrective measures when necessary. The Bottom STAbilization test (BSTA) is used to verify piers have an adequate stabilized bottom (Collin 2007). It involves re-tamping the bottom of the piers to verify that displacement is within acceptable limits. A pattern of successful BSTA tests is sufficient to reduce BSTA verification to spot checks (Fox and Cowell 1998). The Dynamic Cone Penetrometer (DCP) is used in general accordance with ASTM STP 399 *Vane Shear and Cone Penetration Resistance Testing of In situ Soils* to verify aggregate densification within the top few feet of the pier. If average penetration resistance measured consistently exceeds 15 blows, and less than 10% of tests fall below 15 blows per 1.75 inches, then testing may be reduced to spot checks (Fox and Cowell 1998). Modulus testing is used to verify stiffness modulus design assumptions and is based largely on ASTM D1143 *Standard Test Methods for Deep Foundations Under Static Axial Compressive Load*. Typically, one stiffness modulus test is conducted per project site for small projects. On larger projects, between two and four stiffness modulus tests may be conducted. As a general rule, one stiffness modulus test is performed per 1,000 piers (Collin 2007). Uplift tests are conducted when necessary to verify the performance of piers in tension. They are largely based on ASTM D3689 *Standard Test Methods for Deep Foundations Under Static Axial Tensile Load* and generally follow the same load and holding criteria as the modulus test. Often, it is possible to conduct an uplift load test at the same time as the modulus load test, since uplift pier elements are generally used as anchor reactions for the modulus test load frame. All loading and test procedures are available in Collin (2007). Surveying methods should be used to verify pier locations. The center of each pier should be within four inches of the plan location.

Included in the QC procedures should be the completion of daily reports during installation, which include the following information:

- Footing and pier location
- Pier length and drilled diameter
- Planned and actual pier elevations at the top and bottom of the element
- The number of lifts and time of tamping for each lift placed
- Average lift thickness for each pier
- Documentation of soil conditions during drilling for comparison with soil conditions in boring logs
- Depth to groundwater, if encountered
- Documentation of any unusual conditions encountered (e.g., sloughing)
- Type and size of densification equipment used.

QA procedures include monitoring installation of modulus and uplift load test piers, monitoring load tests, performing DCP testing, and monitoring daily pier installation, including observing subsurface conditions and soils during installation. Gradation analyses on samples taken from installed columns may be used to confirm that the in situ gradation matches the specifications and that the columns have not been penetrated by excessive amounts of fines from the surrounding ground. Such testing may be appropriate for the owner's information in a method specification, but columns cannot be rejected for failing to meet a post-installation gradation criterion if other provisions of a method specification have been followed.

5.2.3 Verification Testing

The testing of soils reinforced by aggregate columns should address the different response of the ground when testing granular soils in comparison to predominantly cohesive soils. In situ tests are more appropriate where densification of the in situ soil is anticipated. Load tests are also appropriate for these soils, as well as mixed and cohesive soil profiles. Guidance on the usefulness of certain commonly performed test methods (Esrig and Bachus 1991) is presented in Table 5-1.

Table 5-1. Suitability for Testing Aggregate Columns

Test	Granular	Cohesive	Comments
Dynamic Cone	2	1	Too insensitive to reveal clay lenses. Can locate dense layers and buried features.
Mechanical Cone	3	1	Rarely used.
Electric Cone	4	2	Particle size important. Can be affected by lateral earth pressures generated by treatment. Best test for seismic liquefaction evaluation.
Boreholes + SPT	3	2	Efficiency of test important. Recovers samples.
Dilatometer	3	1	Rarely used.
Pressuremeter	3	1	Rarely used.
Small Plate Load Test	1	1	Does not adequately confine stone column. Affected by pore water pressures.
Large Plate Load Test	2	2	Better confining action.
Zone Loading	4	4	Best test for realistic comparison with foundations.
Full-Scale	5	5	Rare

Note: Suitability ranking varies from 1 as least suitable to 5 as most suitable.

Short duration tests on of 2-foot diameter (small plates in Table 5-1) metal plates are the most common form of testing aggregate columns in Great Britain. This is due to their speed and low cost. However, such tests can only stress the soils to shallow depths and have been susceptible to misinterpretation of actual aggregate column behavior, particularly when residual porewater pressures are present in the ground. To partially get around these obstacles large diameter plate test where the diameter of the plate is equal to the diameter of the column are typically used in the United States.

To overcome these limitations, and to provide more realistic simulation of applied loads, zone loading or dummy footing tests are occasionally performed. Here, loadings of up to 3 times the design bearing pressure are applied over a group of aggregate columns, typically of 4 to 9 in number. Significant expense is involved with these tests. As a result, these tests tend to be performed on larger contracts or where the soil profile is variable; in combination with plate tests to permit correlation between individual aggregate columns and group performance.

It is important that the loaded area be of sufficient dimension and magnitude to induce significant stress into the "critical layer." This stratum is normally the weakest cohesive layer of significant thickness. This layer determines the allowable load of the aggregate column.

6.0 COST DATA

This section presents guidelines for preparing budget estimates in order to determine the economic feasibility of aggregate columns. There are many factors affecting the price of aggregate column construction, including labor, the price and availability of stone, weather, environment, etc. Therefore, it is recommended that experienced contractors with a record of installing aggregate columns be contacted to verify both the budget cost calculations and the technical feasibility of aggregate column installation.

The costs of aggregate columns on a highway project are typically captured in a contract bid item which is measured by the lineal foot (LF). Included in this bid item are the material, equipment, labor, and incidentals to construct an aggregate column. Mobilization associated with the installation of aggregate columns may be measured and paid for separately.

Construction cost items that are associated with aggregate columns, along with approximate cost ranges, are listed in Table 5-2.

Table 5-2. Unit Costs

Pay Item Description	Quantity Range	Unit	Low Unit Price	High Unit Price	Factors that May Impact Costs
Aggregate Columns	Greater than 1,000	LF	$20.00	$60.00	Cost of aggregate materials is sensitive to material specification and haul distance. Unit costs will decrease as total quantity increases. Typical price range is $20 to $40 per lineal foot.
Mobilization	1	Rig	$20,000	$100,000	Mobilization cost increases for distances greater than 500 miles. Phased construction may require multiple mobilizations. High price for rigs for moderate depth treatment is $40,000.
Embankment	Greater than 5,000	CY	Use agency data	Use agency data	Use historical costs that are representative of the project quantity, project conditions and project location
Working Platform Geosynthetic	Greater than 5,000	SY	$1.00	$3.50	Geogrids are more expensive than geotextiles Heavier geotextiles cost more Specified overlap widths impact the total quantity of material required.
Granular Fill Material	Greater than 2,500	Tons	$7.00	$20.00	Material specification and haul cost will impact costs

Cost ranges are based on data (i.e., review of State DOT's bid tabs for aggregate columns) from 2007 through 2010. Readers should carefully examine the project characteristics and constraints and determine to what degree, if any, these factors may influence the actual cost associated with constructing aggregate columns. For many aggregate column applications, a working platform will be required. These costs should be included when comparing this technology with others. The cost of the geosynthetic for the working platform is also provided in this table.

Using the information in the preceding sections, a determination can be readily made as to the depth of the aggregate column installation and the spacing required to satisfy the design intent. The area of treatment should take into consideration the effect of the proposed loading on the soil being improved by aggregate columns. It is recommended that for aggregate column installation the loads be considered as being transmitted on a 45-degree angle around the specified treatment zone perimeter. This will extend the area that requires improvement. A spreadsheet is available at http://www.GeoTechTools.org for performing preliminary budgets for aggregate columns.

7.0 CASE HISTORIES

Representative case histories of transportation-related construction projects are presented to illustrate the application of rammed aggregate pier and stone column technologies.

7.1 Rammed Aggregate Piers Case History

7.1.1 Basic Information

- Project Name: US 90 at SH 6
- Project Location: Sugarland, Texas
- Owner: Texas Department of Transportation
- Engineers: Geotechnical Engineer – HVJ Associates | Structural Engineer: Chiang Patel & Yerby
- Contractor: W.W. Webber, Inc.
- Year Constructed: 2006

7.1.2 Project Summary

This project consisted of ground improvement for support of several MSE walls located at the US90 and SH6 interchange. This was the first MSE wall application that was supported by the rammed aggregate piers that was monitored and instrumented by FHWA's Houston office. The instrumented MSE wall had a maximum height of 27 feet.

7.1.2.1 Subsurface Conditions

The soil conditions consisted of soft to medium stiff clay to 30 feet below ground surface, underlain by sandy silt to silty sand from 30 to 40 feet, overlying sand to silty sand to the maximum explored depth of 60 feet below ground.

7.1.2.2 Technology Used

Rammed aggregate piers provided a cost-effective solution for this MSE wall project saving clients 20 to 50% compared to traditional deep foundation alternatives. Using rammed aggregate piers to reinforce good to poor soils, this ground modification technique allows for visible inspection of the spoils, and the opportunity to address changing ground conditions as they happen. It is an effective replacement for massive over-excavation and replacement or deep foundations, including driven piles, drilled shafts or auger cast-in-place piles. The rammed aggregate pies are constructed by applying direct vertical ramming energy to densely compact successive lifts of high quality crushed rock to form high stiffness engineered elements. The vertical ramming action also increases the lateral stress and improves the soils surrounding the cavity, which results in foundation settlement control and greater bearing pressures for design. Vertical impact ramming results in high density and high strength columns providing superior support capacity, increased bearing pressure up to 10,000 psf and excellent settlement control.

7.1.2.3 The Construction Process

The unique installation process utilizes pre-augering and vertical impact ramming energy to construct rammed aggregate piers, which exhibit high strength and stiffness. The process first involves drilling a cavity. Drill depths normally range from about five to 30 feet, depending on design requirements. Pre-drilling allows you to see the soil between the borings, ensuring that the piers are engineered to reinforce the right soils. Layers of aggregate are then introduced into the drilled cavity in lifts (Figure 5-28).

Courtesy Geopier Foundation Company

Figure 5-28. Rammed aggregate pier installation.

A patented beveled tamper rams each layer of aggregate using vertical impact ramming energy, resulting in high strength and stiffness. The ramming action densifies aggregate vertically and forces aggregate laterally into cavity sidewalls. This results in excellent coupling with surrounding soils and reliable settlement control.

7.1.2.4 Cost Information

The total contract value was $751,946.

7.1.2.5 Solution

A total of 1,411 rammed aggregate piers with spacing that ranged from four to nine feet on-center were installed beneath wall heights of 14 feet or greater (Figure 5-29).

Courtesy Geopier Foundation Company

Figure 5-29. Completed MSE wall supported on rammed aggregate piers.

As a result the factors of safety for bearing resistance instability and global stability were increased to greater than 2.0 and 1.3, respectively as well as allowing rapid pore water pressure dissipation by radial drainage into the columns. Horizontal displacement at the base of the walls was measured to be less than one and a half inches. The modulus test results showed a total movement of 0.69 inches at a stress of more than 22,000 psf, indicating a pier stiffness greater than twice the assumed design value.

7.2 Stone Columns Case History

7.2.1 Basic Information

- Project Name: Route 22
- Project Location: Wadhams, NY
- Year Constructed: 1987

7.2.2 Resources

Sung, J.T. and Ramsey, I.S. (1988). Slope Stabilization by Stone at Wadhams, NY. *Report by Soil Mechanics Bureau*, New York State Department of Transportation, State Campus, Albany, NY.

7.2.3 Project Summary

Stone columns were used to stabilize a 220 foot long slope along New York Route 22 near Wadhams, NY.

7.2.3.1 Subsurface Conditions

Three meter thick layer of silty clay overlying a 10- to 20-foot layer of over-consolidated, soft silty clay. This clay layer is underlain by a layer of silty gravel in which artesian groundwater conditions were encountered. The liquidity index and activity of the clay were 1.0 and 0.5, respectively.

7.2.3.2 Technology Used

A stabilizing berm, shear key, and stone columns were considered. Berm treatment would require additional right-of-way in a wetland area, and shear key would require extensive excavation. Stone columns installed by the dry, bottom feed methods were found to be the most technically feasible, environmentally acceptable, and economic solution.

7.2.3.3 The Construction Process

The stone columns were installed through the soft clays into the gravel layer to intercept the slip plane near the gravel/clay interface at a depth of 16 feet. A photograph during installation is shown in Figure 5-30.

Sung and Ramsey 1988

Figure 5-30. Stone column installation.

7.2.3.4 Performance Monitoring

Prior to construction, slope movement was measured at approximately 1/32-inch per day. During installation, the total movement was 1/8-inch. Eight years after the project completion; little to no additional movement had been recorded.

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