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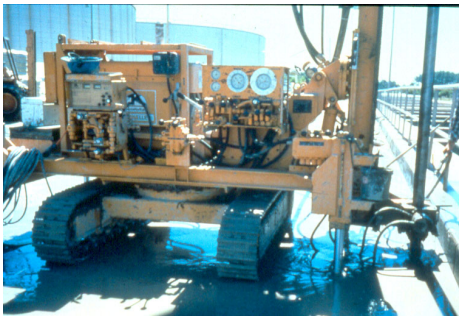
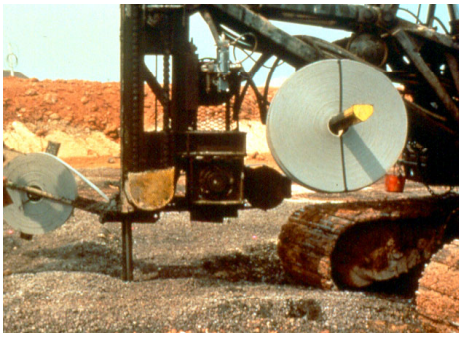
Ground Improvement Methods

Reference Manual – VOLUME I





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16. ABSTRACT This is the reference manual for FHWA NHI course No. 132034 on Ground Improvement Techniques and contains eleven stand alone Technical Summaries on specific ground improvement methods. The summaries provide current practice for design, construction, contracting and quality procedures for the respective technologies. The individual summaries are intended to serve as a primer on a particular subject; and are not intended to present a comprehensive or complete thesis on the technology. Companion, comprehensive technical references for each technology are provided.			
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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	millimeters	0.034	fluid ounces	fl oz
l	liters	0.264	gallons	gal
m^3	cubic meters	35.71	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
tonnes	tonnes	1.103	tons	tons
		TEMPERATURE		
°C	Celsius	1.8 C + 32	Fahrenheit	°F
		WEIGHT DENSITY		
kN/m^3	kilonewton / cubic meter	6.36	poundforce / cubic foot	pcf
	FORCE and PRESSURE or STRESS			
N	newtons	0.225	poundforce	lbf
kN	kilonewtons	225	poundforce	lbf
kPa	kilopascals	0.145	poundforce / square inch	psi
kPa	kilopascals	20.9	poundforce / square foot	psf

PREFACE

One of the major functions of geotechnical engineering is to design, implement and evaluate ground improvement schemes for infrastructure projects. During the last twenty-five 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.

In order to take advantage of these new developments, FHWA originally developed these Technical Summaries (FHWA AS-98-086R) in connection with Demonstration Project No. 116, Ground Improvement Methods. Ground improvement technologies are geotechnical construction methods used to modify and improve poor and marginal soil and rock conditions to meet project requirements. The ground improvement methods addressed in this project include: grouting, vertical consolidation drains, soil mixing, stone columns, lightweight fill materials, vibrocompaction, dynamic compaction, soil nailing, mechanically stabilized earth walls, reinforced soil slopes, micropiles, and column supported embankments.

Implementing one or more ground improvement methods on a project can: increase bearing capacity, control vertical and lateral deformations, decrease imposed loads, provide lateral stability, increase resistance to liquefaction, form a seepage cutoff, fill voids, provide significant cost savings versus alternative construction techniques.

This reference manual is intended for design generalists (project planners, roadway designers, consultant reviewers), design specialists (geotechnical, structural), construction engineers, and specification and contracting specialists involved with projects having problematic site conditions.

Each technical summary reflects current practice in design, construction, contracting methods, and quality procedures. This publication was prepared with the practicing transportation specialist in mind and has been prepared with the benefit of extensive industry review.

These Technical Summaries have evolved from the following FHWA and Industry references:

- Mechanically Stabilized Earth Walls and Reinforced Soil Slopes – Design and Construction Guidelines, by V. Elias, B.R. Christopher and R.R. Berg, FHWA NHI-00-043.
- Manual for Design and Construction Monitoring of Soil Nail Walls, by R. J. Byrne, D. Cotton, J.Portefield, C. Walschlag and G. Ueblaker, FHWA-SA-96-069.
- Dynamic Compaction, Geotechnical Engineering Circular No. 1, by R. Lukas, FHWA- SA-95-037.
- Prefabricated Vertical Drains, Vol. 1, by J.J. Rixner, S.R. Kraemer and A.D. Smith, FHWA-RD-86-168.

- Design and Construction of Stone Columns, Vol. 1, by R. Barksdale and R. Bachus, FHWA-RD-83-02C.
- Lightweight Filling Materials, Permanent International Association of Road Congresses, 1997.
- Geosynthetic Design and Construction Guidelines, by R.D. Holtz, B.R. Christopher and R.R. Berg, FHWA-HI-95-038.

The authors recognize the efforts of Mr. Silas Nichols who was the FHWA Technical Consultant for this update. The authors also recognize the efforts of Mr. Jerry A. DiMaggio, P.E. who was the FHWA Technical Consultant for the Demo 116 and this work, and served in the same capacity for some of the referenced publications. Mr. Nichols' input into this update and Mr. DiMaggio's guidance, editing and input to this and the previous works have been invaluable.

The authors further acknowledge the efforts of the Technical Working Group members of the Demo 116 Project who served as a review panel, listed in alphabetical order:

Dr. Donald Bruce	- ECO Geosystems, Inc.
Dr. James Collin	- The Collin Group
Mr. Albert DiMillio	- FHWA
Mr. Richard Endres	- Michigan DOT
Mr. John Hooks	- FHWA
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Mr. Peter Osborn	- FHWA
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Mr. Michael Simac	- Ground Improvement Technologies
Mr. Ed Tavera	- Louisiana DOT

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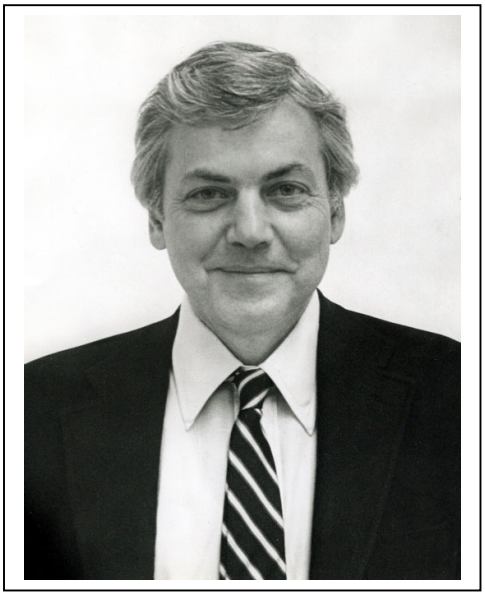

This manual is the last major work of the lead author, **VICTOR ELIAS, P.E.**

Victor Elias was a well-respected Civil Engineer who practiced in the Washington D.C. area for over 35 years. He received his Bachelor of Science and Master of Science degrees from Polytechnic Institute of Brooklyn. He was internationally recognized as an expert in Geotechnical Engineering and was instrumental in the development of national design codes. He was the principal investigator for several national/international research programs and implementation projects.

Mr. Elias had a distinguished professional career and provided significant contributions to the design and construction of safe, cost-effective geotechnical works in highway (and private) works. He has been the Principal Investigator for several major research and/or implementation projects focused on durability of soil reinforcement materials and design specifications for foundations and retaining walls, and ground improvement methods including:

Ground Improvement Methods (2006), Elias, V., Welsh, J., Warren, J., Lukas, R., Collin, J.G. and Berg, R.R. [Technical Consultants - DiMaggio, J.A. and Nichols, S.], FHWA NHI-06-019 (Vol. I) and FHWA NHI-06-020 (Vol. II), 1056 p.

Geotechnical Engineering Circular # 7, Soil Nail Walls (2003), Lazarte, C., Elias, V., Espinosa, D. and Sabatini, P., FHWA IF-03-017, 283 p.

Mechanically Stabilized Earth Walls and Reinforced Soil Slopes, Design & Construction Guidelines (2001), Elias, V., Christopher, B.R. and Berg, R.R., [Technical Consultant - DiMaggio, J.A.], FHWA-NHI-00-043, 418 p.

Corrosion/Degradation of Soil Reinforcements for Mechanically Stabilized Earth Walls and Reinforced Soil Slopes (2000), Elias, V., [Technical Consultant - DiMaggio, J.A.], FHWA-NHI-00-044, 94 p.

Testing Protocols for Oxidation and Hydrolysis of Geosynthetics (1999), Elias, V., Salman, A., Juran, I., Pearce, E., Lu, S., FHWA RD-97-144

Ground Improvement Technical Summaries (1998), Elias, V., Welsh, J., Warren, J., and Lukas, R., FHWA SA-98-086, 757 p.

Geotechnical Engineering Circular #27, Earth Retaining Systems (1997), Sabatini, P.J., Elias, V., Schmertmann, G.R. and Bonaparte, R., FHWA-SA-96-038, 161 p.

Stress Crack Potential of HDPE Geogrids, Elias, V., Carlson, D., Bachus, R., and Giroud, J.P., FHWA RD-97-142.

Mechanically Stabilized Earth Walls and Reinforced Soil Slopes, Design & Construction Guidelines (1997), Elias, V. and Christopher, B.R. [Technical Consultants - DiMaggio, J.A and Berg, R.R.,] FHWA-SA-00-071, 371 p.

Manual for Design and Construction Monitoring of Soil Nail Walls (1996), Elias, V., et al., FHWA SA-96-069.

Corrosion/Durability of Soil Reinforced Structures (1989), Elias V., FHWA RD 89-186, 105 p.

NCHRP 12-35: Recommended Revisions to Sections 4, 5 and 7 of AASHTO Standard Specifications for Highway Bridges**,** V. Elias Co-Principal Investigator, National Academy of Sciences.

Mr. Elias was instrumental in the introduction and implementation of reinforced soil technology in the U.S., as a Vice President for The Reinforced Earth Company from 1974 to 1985. His work included major reinforced soil retaining walls. In addition, he expanded the applications of soil reinforcement to slabs, dams, storage facilities and bridge abutments.

In 2002 in Nice, France, the International Geosynthetics Society recognized Mr. Elias for his seminal work on the durability of geosynthetics. In 1997, he was awarded the North American Geosynthetics Society Award of Excellence for his outstanding contribution to geosynthetic research and development. In 1993, he received the T. Allan Haliburton Medal for his work with the American Society of Testing Materials.

His many colleagues, fellow engineers and friends within the geotechnical community will dearly miss Victor's leadership, insights, staunch opinions, and experience.

Engineers designing highway geotechnical works will be referring to Mr. Elias' publications for the foreseeable future.

Jerry A. DiMaggio, P.E.
Principal Geotechnical Engineer
Federal Highway Administration
Washington, D.C.

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INTRODUCTION

Ground improvement technologies are geotechnical construction methods used to alter and improve poor ground conditions in order that embankment and structure construction can meet project performance requirements, where soil replacement is not feasible for environmental or technical reasons, or it is too costly.

Ground improvement has one or more than one of the following main functions:

- To increase bearing capacity, shear or frictional strength,
- to increase density,
- to control deformations,
- to accelerate consolidation,
- to decrease imposed loads,
- to provide lateral stability,
- to form seepage cutoffs or fill voids,
- to increase resistance to liquefaction and,
- to transfer embankment loads to more competent layers

There are three strategies available to accomplish the above functions representing different approaches. The first method is to increase the shear strength, density and/or decrease the compressibility of the foundation soil. The second method is to utilize a lightweight fill embankment to reduce significantly the applied load to the foundation and the third method is to transfer loads to a more competent deeper layer.

The purpose of these Technical Summaries is to introduce available ground improvement methods and applications primarily to generalists involved in project development. The summaries outline methods, function, applications, benefits, limitations and summarize design issues as well as factors influencing selection and contracting methods. Each summary also contains case histories of successful implementation.

For the geotechnical specialist, state-of-the-art design and construction references are provided at the end of each Technical Summary.

The selection of candidate ground improvement methods for any specific project follows a sequential process. The steps in the process include a sequence of evaluations that proceed from simple to more detailed, allowing a best method to emerge. The process is described as follows:

1. *Identify potential poor ground conditions, their extent and type of negative impact*. Poor ground conditions are typically characterized by soft or loose foundation soils which under load would cause long term settlement, or cause construction or post construction instability.

2. *Identify or establish performance requirements.* Performance requirements generally consist of deformation limits (horizontal and vertical), as well as some minimum factors of safety for stability. The available time for construction is also a performance requirement.

3. *Identify and assess any space or environmental constraints.* Space constraints typically refer to accessibility for construction equipment to operate safely and environmental constraints may include the disposal of spoil (hazardous or otherwise) and the effect of construction vibrations or noise.

4. *Assessment of Subsurface conditions*. The type, depth and extent of the poor soils must be considered as well as the location of the ground-water table. It is further valuable to have at least a preliminary assessment of the shear strength and compressibility of the identified poor soils.

5. *Preliminary Selection*. Preliminary selection of potentially applicable method(s) 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.

   Table 1, which groups the available methods in six broad categories, can be used as a guide in this process to identify possible methods and eliminate those that by themselves or in conjunction with other methods cannot produce the desired performance.

6. *Preliminary Design*. A preliminary design is developed for each method identified under Preliminary Selection and a cost estimate prepared on the basis of data in Table 2. The guidance in developing preliminary designs is contained within each Technical Summary.

7. *Comparison and Selection*. The selected methods are then compared and a selection made by considering performance, constructability, cost and other relevant project factors.

For the geotechnical specialist, state-of-the-art design and construction methods and/or references are provided in each Technical Summary to form the basis of a Final Design.

Table 1. Ground Improvement Categories, Functions, Methods and Applications.

Category	Function	Methods	Comment
Consolidation	Accelerate consolidation, increase shear strength	(1) Prefabricated Vertical Drains (PVD) (e.g., Wick drains) (2) Vacuum consolidation	Viable for normally consolidated clays. Vacuum consolidation viable for very soft clays. Can achieve up to 90% consolidation in a few months.
Load Reduction	Reduce load on foundation, reduce settlement	(1) Geofoam, (2) Foamed concrete (3) Lightweight granular fills, tire chips, etc.	Density varies from 1 – 12 kN/m^3 (6 – 76 lb/ft^3). Granular fills usage subject to local availability.
Densification	Increase density, bearing capacity and frictional strength of granular soils. Decrease settlement and increase resistance to liquefaction.	(1) Vibro-compaction using vibrators (2) Dynamic compaction by falling weight impact.	Vibrocompaction viable for clean sands with <15% fines. Dynamic compaction limited to depths of about 10m, but is applicable for a wider range of soils. Both methods can densify granular soils up to 80% Relative Density. Dynamic compaction generates vibrations for a considerable lateral distance.
Reinforcement	Internally reinforces fills and/or cuts. In soft foundation soils, increases shear strength, resistance to liquefaction and decreases compressibility.	(1) MSE retaining walls (2) Soil Nailing walls (3) Stone column to reinforce foundations.	Soil Nailing may not applicable in soft clays or loose fills. Stone columns applicable in soft clay profiles to increase global shear strength and reduce settlement.

Table 1. Ground Improvement Categories, Functions, Methods and Applications (cont.)

Category	Function	Methods	Comment
Chemical Stabilization by Deep Mixing Methods	Physio-chemical alteration of foundation soils to increase their tensile, compressive and shear strength, and to decrease settlement and/or provide lateral stability and or confinement.	(1) Wet mixing methods using primarily cement (2) Dry mixing methods using lime-cement	Applicable in soft to medium stiff clays for excavation support where the groundwater table must be maintained or for foundation support where lateral restraint must be provided or to increase global stability and decrease settlement. Requires significant QA/QC program for verification
Chemical Stabilization by Grouting	To form seepage cutoffs, fill voids, increase density, increase tensile and compressive strength	(1) Permeation grouting with particulate or chemical grouts (2) Compaction grouting (3) Jet grouting, and (4) Bulk filling	(1) Permeation grouting to increase shear strength or for seepage control, (2) compaction grouting for densification and (3) jet grouting to increase tensile and/or compressive strength of foundations, and (4) bulk filling of any subsurface voids
Load Transfer	Transfer load to deeper bearing layer	Column (Pile) supported embankments on flexible geosynthetic mats	Applicable for deep soft soil profiles or where a tight schedule must be maintained. A variety of stiff or semi-stiff piles can be used.

Table 2.a Comparative Costs (SI Units)

Method	Unit Cost	Cost of Treated Volume $/m^3
Prefabricated Vertical Drains (PVD)[a] (e.g., Wick drains)	$0.90 - $4.00/m	$ 0.60 - $5.00
Lightweight Fill		
Granular	$3 - $21/m^3	$3 - $21
Tires-Wood	$12 - $30/m^3	$12 - $30
Geofoam	$40 - $85/m^3	$40 - $85
Foamed Concrete	$40 - $55/m^3	$40 - $55
Vibrocompaction[b]	$15 - $25/m	$1 - $4
Dynamic Compaction	$8 - $25/m^2	$2 - $3
MSE walls	$160 - $300/m^2	
RSS Slopes	$110 - $260/m^2	
Soil Nail walls	$400 - $600/m^2	
Stone Columns[c]	$45 - $60/m	
Deep Soil Mixing		
Dry w/lime-cement	$30/m	$60
Wet w/cement	--	$65 - $150
Grouting		
Permeation[d]	$ 65/m + $0.65/liter	
Compaction		$5 - $50
Jet		$150 - $300
Column Supported Embankments	$ 95/m^2 + cost of column	n/a

Notes:

a. Usually add mobilization/demobilization charge of $8,000 to $10,000, as well as the cost of drainage blanket and instrumentation.
b. $15/lm when no backfill is placed around the probe and $25/lm when granular backfill is added. In addition to unit cost, a mobilization/demobilization cost of $15,000 per rig should be assumed.
c. The minimum cost for vibro-replacement stone column installation, based on readily available suitable backfill material, is $45 per linear meter, with a dry vibro displacement stone column starting at $60 per meter. In addition to unit cost, a minimum mobilization/demobilization cost of $15,000 per rig should be assumed.
d. Add mobilization/demobilization charge of $10,000 to $50,000.

The success of any ground improvement method is predicated on the implementation of a QA/QC program to verify that the desired foundation improvement level has been reached. These programs incorporate a combination of construction observations, in-situ testing and laboratory testing to evaluate the treated soil in the field. Details are provided in each Technical Summary.

Lists of ground improvement contractors are appended (to Volume II).

Table 2.b Comparative Costs (U.S. Customary Units)

Method	Unit Cost	Cost of Treated Volume $/yd^3
Prefabricated Vertical Drains (PVD)[a] (e.g., Wick drains)	$0.30 - $1.25/ft	$0.45 - $4
Lightweight Fill Granular Tires-Wood Geofoam Foamed Concrete	 $2.30 - $16/yd^3 $9 - $23/yd^3 $30 - $65/yd^3 $30 - $42/yd^3	 $2.30 - $16 $9 - $23 $30 - $65 $30 - $42
Vibrocompaction[b]	$ 4.50 - $7.0/lft	$0.75 – $3
Dynamic Compaction	$0.75 - $2.30/ft^2	$1.50 - $2.50
MSE walls	$ 15.00 - 28.00/ft^2	
RSS Slopes	$ 10.00 - 24.00/ft^2	
Soil Nail walls	$ 37.00 - 56.00/ft^2	
Stone Columns[c]	$14 - $18/lft	
Deep Soil Mixing Dry w/lime-cement Wet w/cement	 $100/ft --	 $50 $55 - $125
Grouting Permeation[d] Compaction Jet	$ 20/ft + $ 2.50/gallon	 $4 - $40 $115 - $230
Column Supported Embankments	$ 9/ft^2 + cost of column	n/a

Notes:

a. Usually add mobilization charge of $8,000 to $10,000, as well as the cost of drainage blanket and instrumentation.
b. $15/lm when no backfill is placed around the probe and $25/lm when granular backfill is added. In addition to unit cost, a mobilization/demobilization cost of $15,000 per rig should be assumed.
c. The minimum cost for vibro-replacement stone column installation, based on readily available suitable backfill material, is $14/lft., with a dry vibro displacement stone column starting at $18/lft. In addition to unit cost, a minimum mobilization/demobilization cost of $15,000 per rig should be assumed.
d. Add mobilization/demobilization charge of $10,000 to $50,000.

Technical Summary # 1:

CONSOLIDATION USING VERTICAL DRAINS

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CHAPTER 1
DESCRIPTION AND HISTORY

1.1 DESCRIPTION

Consolidation of soft ground using vertical drains is a technique used since the 1920s. Initially sand drains were used, then cardboard drains, followed by geotextile-encased drains (wick drains), and finally vacuum consolidation, which was developed as a companion technology suitable for consolidating extremely soft soils of significant depth in a potentially shorter time frame. The most common method at present is the use of *wick* drains, which can be best described as prefabricated vertical drains (PVDs), 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. This construction permits pore water in the soil to seep into the drain for collection and transmittal up and down the length of the core. 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 wick drains are prefabricated vertical drains or PVDs, drainage wicks, band drains, and strip drains (although the term strip drain also refers to other prefabricated composite drains).

While there are some variations, the size of a PVD is typically 100 mm (4 in.) wide by 3-9 mm (⅛-⅜-in.) thick. The material consists of a plastic core formed to create channels that are wrapped in a geotextile filter.

PVDs are only one general type of vertical drain system, the others being wick drains, sand drains, and "sand wicks," or geotextile-encased sand drains. Stone columns (see Technical Summary # 5) also provide vertical drainage. The general principles that govern all vertical drain installations are similar to 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 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 is not acceptable. A typical cross-section of a PVD installation for embankments is shown in Figure 1.

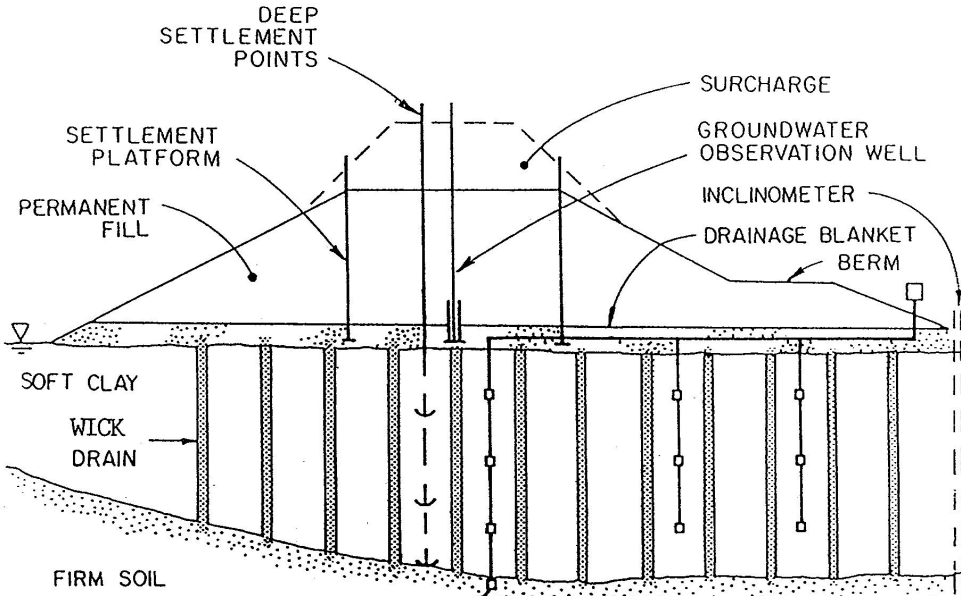


Figure 1. Prefabricated Vertical Drain Installation for a Highway Embankment.

A preload and surcharge are shown in Figure 1. Preload is defined as 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 added fill above final elevations that is used to accelerate settlement or minimize secondary consolidation.

Consolidation of compressible soils by vacuum 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 sealed soil to be consolidated and maintaining the vacuum during a predetermined period of time. The soil is, therefore, loaded uniformly throughout its depth by the equivalent of one atmosphere as shown in Figure 2.

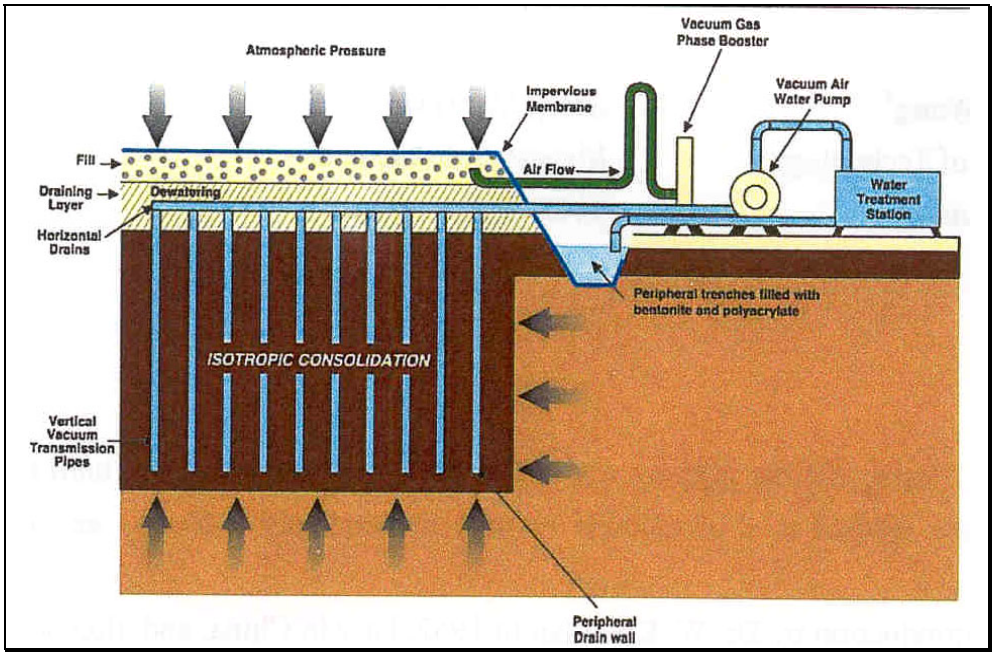


Figure 2. Vacuum System Typical Cross-Section.[7]

1.2 HISTORICAL OVERVIEW

The development of PVDs almost parallels the development of the vertical sand drain concept. A U.S. patent for a sand drain system was granted in 1926 to D. 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 100 mm (4 in.) by 3 mm (⅛ in.) 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 geosynthetic 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, quality vertical drain system. Another contributing factor to this timing was that all of the practical 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 as many as 200 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 different PVDs used worldwide and at least 10 in the United States. Today, most of those installed in the United States are one of two basic designs. Both have an independent outside geosynthetic 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 over 90 percent of the United States market. Commonly used brand names include AMERDRAIN 407, MEBRA-DRAIN 7407, MEBRA-DRAIN MD 88, and ALIWICK.

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 10 percent of the United States market because of their slightly higher cost.

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. Representative PVDs in use are shown in Figure 3.

Vacuum consolidation achieved a greater measure of reliability in the last decade, as many of the technological problems associated with the method were identified and solved. In principle, vacuum increases the effective stress by one atmosphere in all directions, because

this increment is transmitted through the water phase, which is isotropic. In practice, the increase is smaller and principally a function of the efficiency of the vacuum pumps and the ability to seal the system periphery. [7]

Over two dozen vacuum consolidation projects have been completed worldwide in the last decade; however, no commercial project has been completed to date in the United States.

1.3 FOCUS AND SCOPE

The principal purpose of this technical summary is to acquaint the reader with the use of vertical drain methods. Many of the concepts and ideas stated are taken from practical experience and the basic references, listed below:

1. "Prefabricated Vertical Drains," *U.S. Department of Transportation, Federal Highway Administration, Research, Development, and Technology, Vol. I: Engineering Guidelines, Report No. FHWA/RD-86-168*.[1]

2. "Shared Experience in Geotechnical Engineering: Wick Drains," *Transportation Research Circular, Number 309, September 1986, ISSN 0097-8515*.[2]

For more specific information, the reader should refer to these references of this technical summary.

The content of this technical summary includes the following:

- Application
- Feasibility
- Construction procedures and monitoring
- 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.

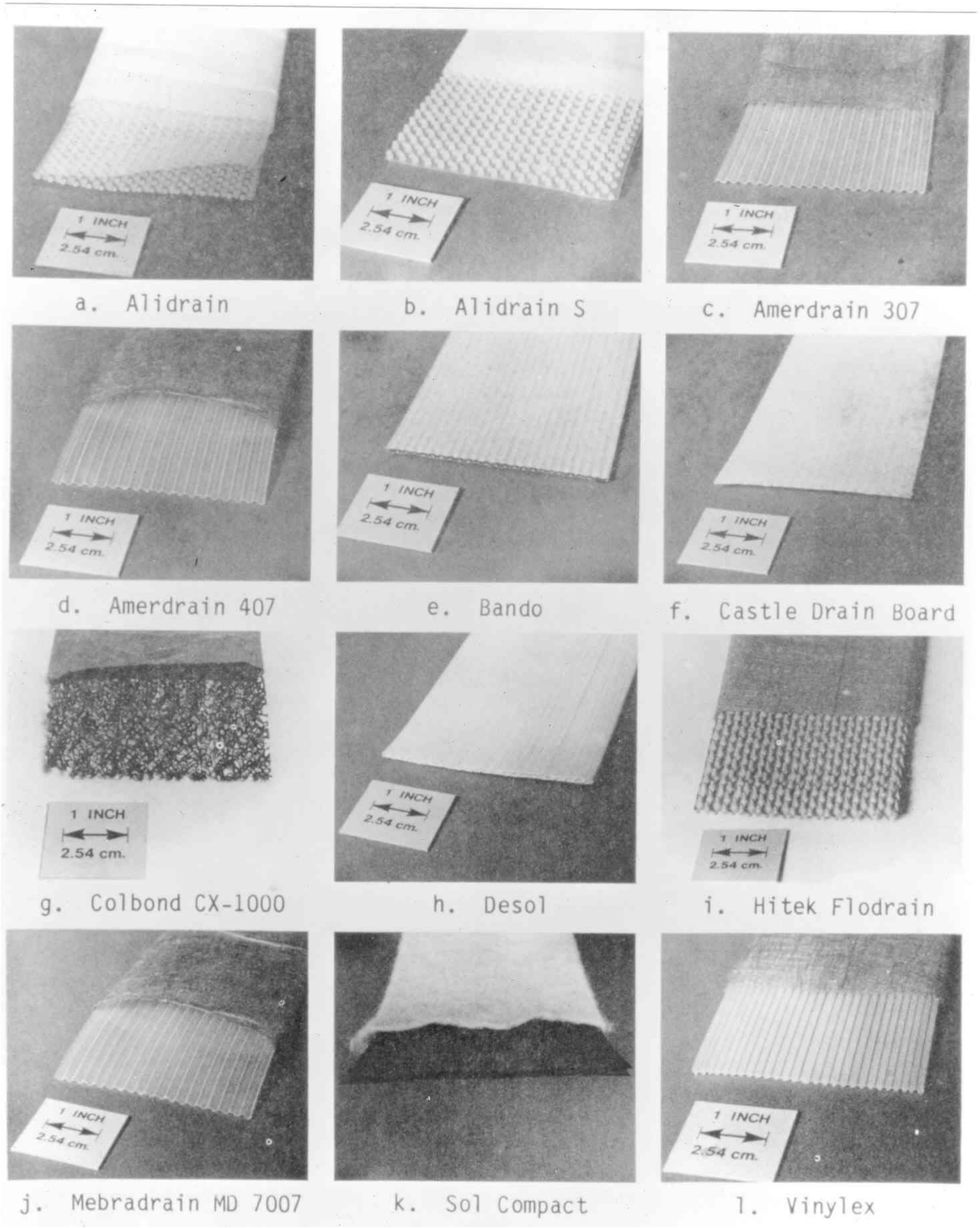


Figure 3. Typical Prefabricated Vertical Drain Products.

When used in conjunction with surcharging or preloading, the principal benefits of using PVDs are

- 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.

Vacuum consolidation may be considered when the compressible deposits are quite deep, the subsoils are very soft, the stage or final embankment height is no greater than about 3.5 m (11.5 ft.), and/or the placement of surcharge to eliminate secondary compression is impractical because of cost, available surcharge time, or space limitations.

CHAPTER 2
FEASIBILITY CONSIDERATIONS

2.1 APPLICATIONS

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 fully saturated in its natural state. Such soils are typically described as silts, clays, organic silts, organic clays, muck, peat, swamps, muskeg, and sludge.

In general, soils for which PVDs are being used must be saturated and normally to slightly over-consolidated prior to loading. The loading should exceed the maximum past consolidation pressure 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.

Table 1 illustrates many of the common uses of PVDs, but is not to be considered all-inclusive.

Vacuum Consolidation

As previously indicated, vacuum consolidation is primarily applicable where the subsoil is very soft, typically organic, and where time for stage construction is not available. Vacuum consolidation may be combined with surcharge by placing additional fill material on top of the impervious membrane. A benefit is that the application of vacuum induces an apparent cohesion in the sand blanket due to the increase of effective stress. This increase of strength in the granular layer provides an additional resisting moment in stability analyses permitting larger stage heights.[9]

Special Application, Liquefaction Reduction

One of the potential uses of PVDs that has not yet been fully explored is the installation of drains to reduce the potential for liquefaction. There has been at least one project completed in the United States where this has been the sole purpose. This was a tank storage project overlying silty sands with numerous intervening silt layers inhibiting vertical drainage.

Table 1. Uses of PVDs for transportation applications.

Field of Application	Purpose	
	Increase Stability	Accelerate Settlements
Highways		
• Roadways	X	X
•Structure Approach Fills	X	X
Airfields		
•Runways	X	X
•Taxiways	X	X
Earth Dams		
•Embankments	X	X
Storage Tanks	X	X
Pile Foundations		
•Reduce "Negative" Skin Friction		X
Marine Walls	X	X
Large-Scale Development of Marginal Areas - Regrading	X	X

PVDs provide two potential benefits that will reduce the possibility of liquefaction in susceptible soils: drainage and reinforcement. There has been minimal research to determine the extent of potential liquefaction reduction due to PVDs. However, because PVDs offer a pore pressure relief mechanism and have some tensile strength, it appears that some degree of improvement should be obtained.

2.2 ADVANTAGES AND DISADVANTAGES

The advantages and disadvantages of a PVD solution are discussed as a comparison with other vertical drain solutions, specifically sand or other aggregate drains. Alternate site improvement methods to PVDs are subsequently indicated.

a. Advantages

Economy

For typical projects, the cost for PVDs will be one-fifth to one-tenth that of 300-450 mm (1-1.5 ft.) sand drains. Site conditions could affect this differential, but more than likely would affect the pricing of both type drains. For projects that are extremely small (less than 1,500 linear meters), sand drains may be more economical due to high mobilization charges for PVD equipment. It should be noted that PVD spacing would be 1/2 to 1/3 that of sand drains. Hence, the cost savings are not as attractive as presented above.

Installation

Generally, the production rate for PVDs will average between 3,000-4,500 meters (10,000-15,000 ft.) per day per rig. Production rates as high as two to three times these figures have been achieved, but for planning purposes, 3000 meters (10,000 ft.) per day should be used. Sand drains would have a production rate of less than one-fifth that of the PVDs. However, the production rate is balanced as compared to PVDs by the fact that approximately 2 to 3 times more PVDs are typically needed.

Continuity of Drain

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

Minimal Displacement

The typical size of mandrel used for PVDs is small enough to create minimal displacement during installation. However, when compared with high-quality, non-displacement sand drains, PVDs might not be quite as effective. The amount of displacement 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 value of in-situ soils due to remolding as compared with displacement sand drains.

Improved Quality Control

Compared to sand drains, 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.

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.

Less Material Storage

PVDs come in reels usually containing 140-300 meters (450-1,000 ft.). Each roll is approximately 1 meter (3.3 ft.) in diameter, 100 mm (4 in.) thick, and can be easily stored. PVDs will eliminate the cost of large sand stockpiles and truck traffic usually associated with sand drains.

No Spoil Removal

With the exception of situations where pre-augering or drilling is necessary to penetrate to the soft compressible soil layer, there is no significant excess spoil material to be removed from the site. By comparison with jetted sand drains, PVDs do not require the handling and disposal of contaminated water or jetted spoil materials.

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.

b. Disadvantages

Greater Quantities

Because of their size, PVDs need to be placed at a closer spacing than sand drains. A rule of thumb is that the quantity of PVDs to cover the same area will be 2 to 3 times that of 300-mm (11.8-inch) mandrel-driven sand drains. Even with this differential, the total cost of a

PVD solution is usually substantially less than sand drains, as the installation rate on an area basis is much greater.

No Compressive Strength

While sand drains do exhibit a small amount of added compressible strength at fairly close centers, PVDs do not.

Headroom Limitations

Generally, PVD installation equipment must be 1.5-3 meters (5-10 ft) taller than the depth of installation. A headroom requirement for sand drains is at least 3 meters (10 ft.) more than the depth of installation.

Material 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 geosynthetic 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, the subsoil must meet the following criteria:

- Moderate to high compressibility.
- Low permeability.
- Full saturation.
- Final embankment loads must exceed maximum past pressure.
- 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 site conditions.

a. Geotechnical Criteria

To ascertain if the geotechnical criteria to ensure successful design can be met, 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 are extremely useful in determining strata changes and provide 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 strength. 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.[3]

b. 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.

c. 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 45 m (150 ft.)
- 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 8 meters (26 ft.) 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 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 3 meters (10 ft.) 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 (600-900 mm (2-3 ft.) 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 60 meters (200 ft.) with the use of specialized equipment. A rule of thumb is that PVDs over 30 meters (100 ft.) in depth will require a crane for installation. Depths over 35 meters (115 ft.) 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 angle 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 (stage construction), monitored, or if the final or stage fill height is less than 3.5 m (12 ft.), 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.[16] 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. Although they have been installed up to 60 meters (200 ft.) in depth, the use of PVDs to depths greater than 45 meters (150 ft.) should be evaluated by a specialist.

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. References (1) and (15) 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 improvement 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 structural foundations.
3. Reduce the compressibility of the in-situ soil.
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.

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 (CSE). Such solutions are usually much more expensive and may require significant future maintenance. They 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 Technical Summary #5.)
- *Deep soil mixing.* Used to change the in-situ compression and strength characteristics of soils. (See Technical Summary #6.)
- *Excavation* and replacement of compressible soil.
- *Lightweight fills.* Used to reduce the embankment load. (See Technical Summary #2.)

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.

CHAPTER 3
CONSTRUCTION AND MATERIALS

3.1 CONSTRUCTION

a. Prefabricated Vertical Drains

Table 2 lists the various methods of prefabricated vertical drain installation, which employ mostly the same principles. With few exceptions, all methods in Table 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.

Table 2. Methods of prefabricated vertical drain installation.

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

* *Presently used in the United States*

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, or static-vibratory.

Figure 4 illustrates typical installation equipment and carrier pieces. The typical mandrel used in the United States has either a diamond shape of approximately 140 mm x 62 mm (5.5 in. x 2.4 in.) or a rectangular shape with dimensions approximately 50 mm x 130 mm (2.0 in. x 5 in.). 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 13-mm (0.5-in.) diameter piece of cable or rebar or a special anchor plate made of sheet metal. The rebar and cable anchors will be approximately 200 mm (8 in.) 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.

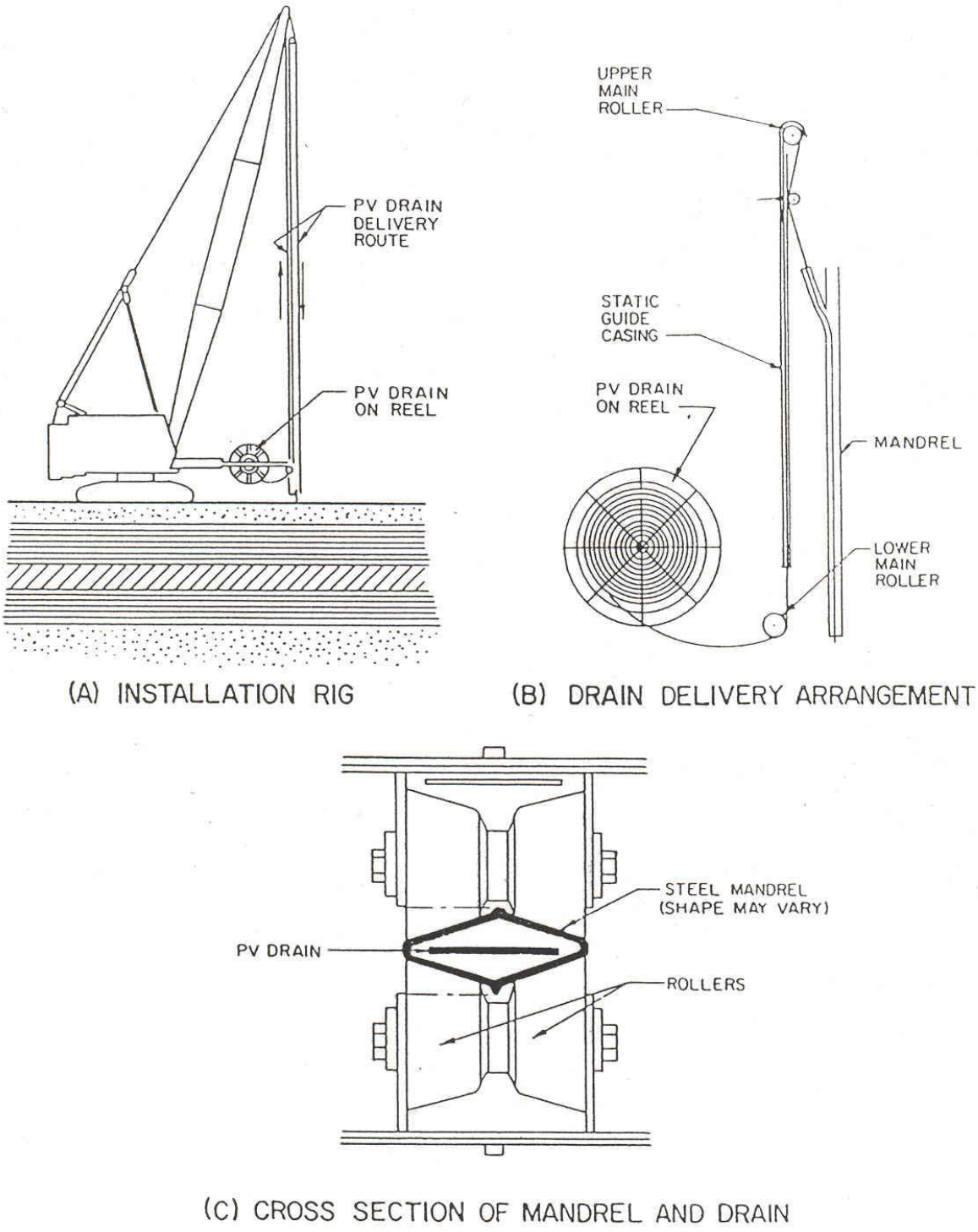


Figure 4. Typical Prefabricated Vertical Drain Installation Equipment.

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 (over 45 m (150 ft.) 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 end of the old roll into the new roll a minimum length of 150 mm (6 in.). 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. Figures 5 and 6 illustrate the construction procedures.

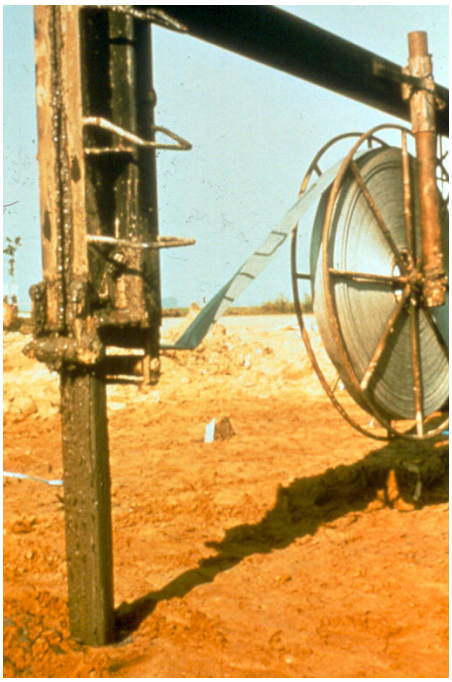
Specialized Equipment

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 (45 meters (150 ft.) 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.

a. Placing the anchor on the drain.

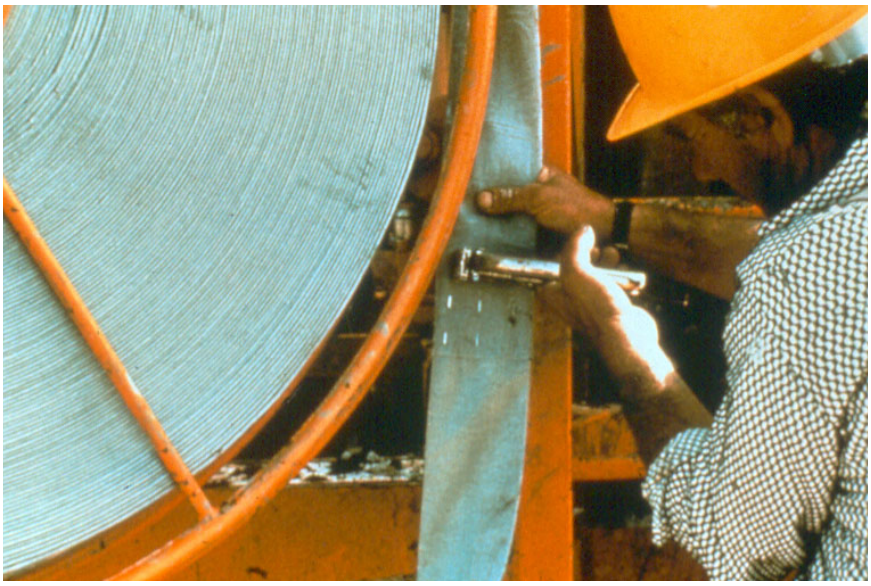


b. Inserting the mandrel into the ground.

c. Cutting the drain after withdrawing the mandrel.

Figure 5. Typical Prefabricated Vertical Drain Installation Procedure.
(*Photographs provided by Geotechnics Holland, BV.*)

a. Inserting the drain core within the jacket to maintain continuity.



b. Stapling the drain splice.

Figure 6. Typical Prefabricated Vertical Drain Splicing Procedure.
(*Photographs provided by Geotechnics Holland, BV.*)

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.

b. Vacuum Consolidation

Figure 2 illustrates typical elements of a vacuum system. The installation procedure is divided into several phases or sub-phases, depending on site conditions, and is designed to meet site-specific objectives. The phases, equipment, methods, and objectives are described below: [7]

- Site preparation. The objective is to construct a stable working platform and a drainage layer. This is accomplished by placement of a geotextile of appropriate strength and a 1 meter (3 ft.) sand layer.
- Drainage. The objective is to transmit vacuum to the soil being consolidated via cylindrical vacuum transmission pipes (drains) connected to horizontal drains inside the sand blanket, which in turn are connected to vacuum pumps. The vacuum transmission pipes are installed in a grid arrangement varying from 0.5-1.5 meters (1.6-5 ft.) on center. To reduce drainage paths, additional PVDs can be installed. Normal drilling equipment is used to install the drains.
- Air and water tightness. The objective is to maintain air and water tightness within any upper sandy strata and/or to the water table. This may be accomplished by constructing slurry walls and/or slurry trenches with impervious membranes to the water table.
- Primary fill. The objective is to increase stability of the system during the vacuum phase. This is accomplished by constructing 1.5 m (5 ft.) of fill above the sand layer and under the impervious membrane.
- Membrane and pumping system. The objective is to establish a vacuum pumping efficiency of 75 percent. Welding the membrane and exiting the horizontal drains at the vacuum pumping stations accomplishes this objective. Vacuum pumping then proceeds until the consolidation objectives are achieved.

After the consolidation objectives under vacuum have been achieved, additional stage fill may be constructed.

3.2 MATERIALS OF CONSTRUCTION

a. Prefabricated Vertical Drains

PVDs are relatively flat and approximately 100 mm (4 in.) wide by 3-9 mm (⅛-⅜ in.) thick. The material generally consists of a plastic core formed to make channels and a 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.

b. 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 0.074 mm (#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 300 mm (1 ft.), but more typically it is 0.5-1 meter (1.6-3 ft.). Where it is used directly over soft soils, typical designs require at least 300 mm (1 ft.) of extra thickness because of potential contamination, or use of a geotextile separator.[15] Where there is a possibility of mud waving, as much as a 1-meter (3 ft.) 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 200 mm (8 in.), 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 25 mm (1 in.) in thickness and either 150 mm or 300 mm (0.5 ft. or 1 ft.) 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 strip drain application is shown in Figure 7.

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.

c. Vacuum Consolidation

The equipment and materials required to ensure a successful application are based on experience and should be designed, supplied, and constructed by specialty contractors with a successful track record with this technology.

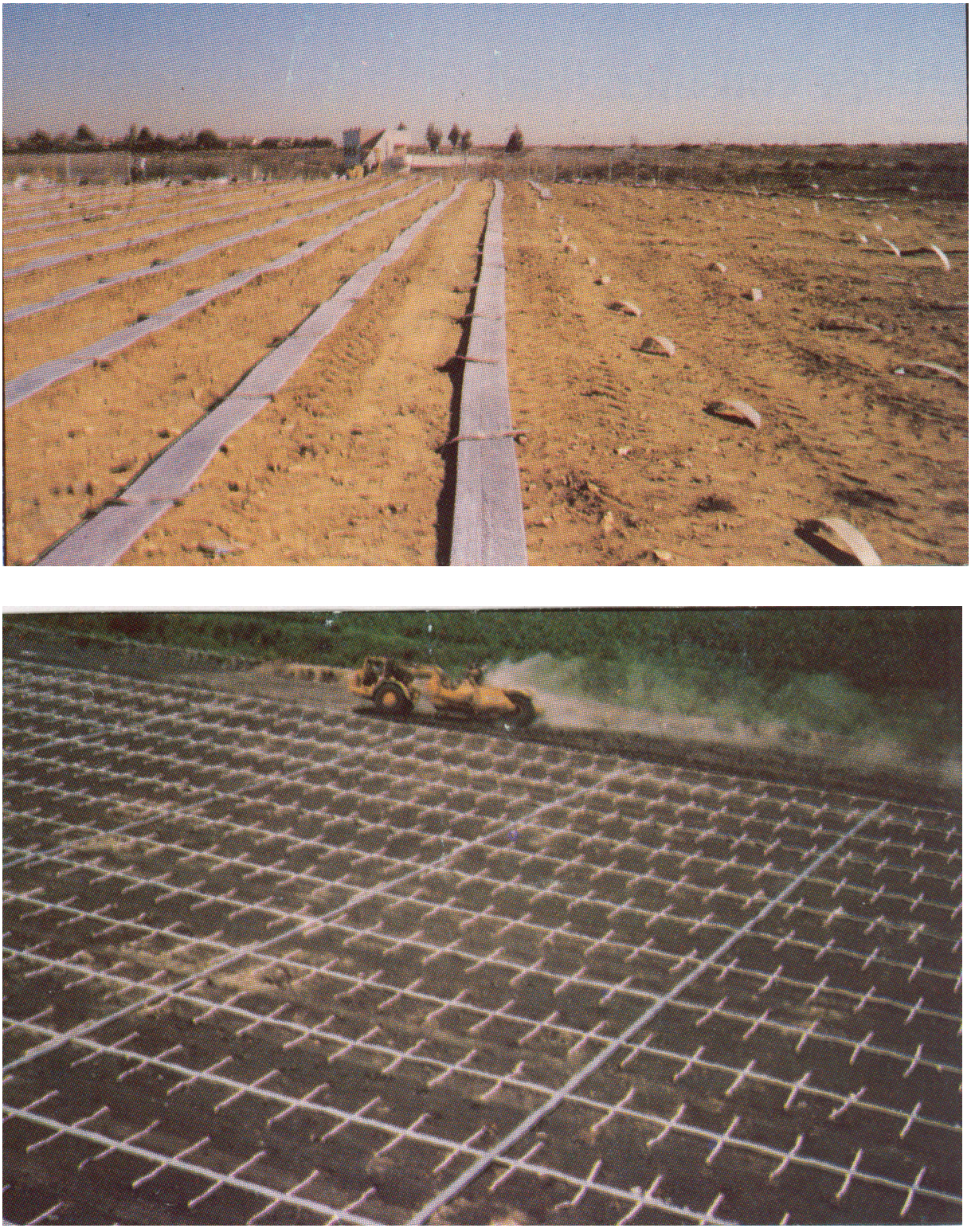


Figure 7. Strip Drains.

CHAPTER 4
DESIGN CONCEPTS

Design is primarily based on the use of vertical drains to reduce the length of the drainage path and, thereby, accelerate consolidation. Vacuum consolidation may also use PVDs to shorten drainage paths and further reduce consolidation time. For unusual applications, such as reinforcement, liquefaction potential reduction and/or water table changes, specialized knowledge of the function of the PVDs will be necessary and are beyond the scope of this technical summary.

For detailed design procedures, the reader should consult the following references for specifics of the design methods:

- "Prefabricated Vertical Drains – A Design and Construction Guidelines Manual," *Federal Highway Administration*. [1]

- "Soil Mechanics." NAVFAC Design Manual 7.1, *Department of the Navy, Naval Facilities Engineering Command*.[8]

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 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) 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. (See Reference (18) for detailed discussions on SPT and PCPT.)

4.1 DESIGN CONCEPTS

The principal purpose of any PVD design is 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 and the time rate settlement under the final embankment load. PVDs may be a design solution if the time to reach 90-95 percent of these projected settlements is too great and beyond the contract construction time.

The assumptions used in developing one dimensional consolidation theory were 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.[6, 11]

$$t = \frac{D^2}{8c_h}(F(n) + F_s)\ln\left[\frac{1}{1-\overline{U_h}}\right] \quad (1)$$

where:

t = time required to achieve desired average degree of consolidation

$\overline{U_h}$ = average degree of consolidation due to horizontal drainage

D = 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

$$= \ln\left(\frac{D}{d}\right) - 0.75 \text{ (simplified)} \quad (2)$$

d = diameter of an equivalent circular drain

F_s = Factor for soil disturbance

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

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

- **F_s**. The above basic relationship can be modified further to consider the effects related to well resistance. While the effects of both soil disturbance and well resistance are often ignored for typical projects, the reader should consult References (1) and (3), in order to fully evaluate and design for those effects.

  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 1.5 m (5 ft.) 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 (10), by Hansbo.

- **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 1.2-1.5 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.

  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 Asoaka method shown in Figure 8 and discussed in Reference (3).

Equation 1 does not take into consideration any consolidation from vertical drainage. The vertical drainage effect may be minor when the compressible layer is over 5 meters (16 ft.) 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.

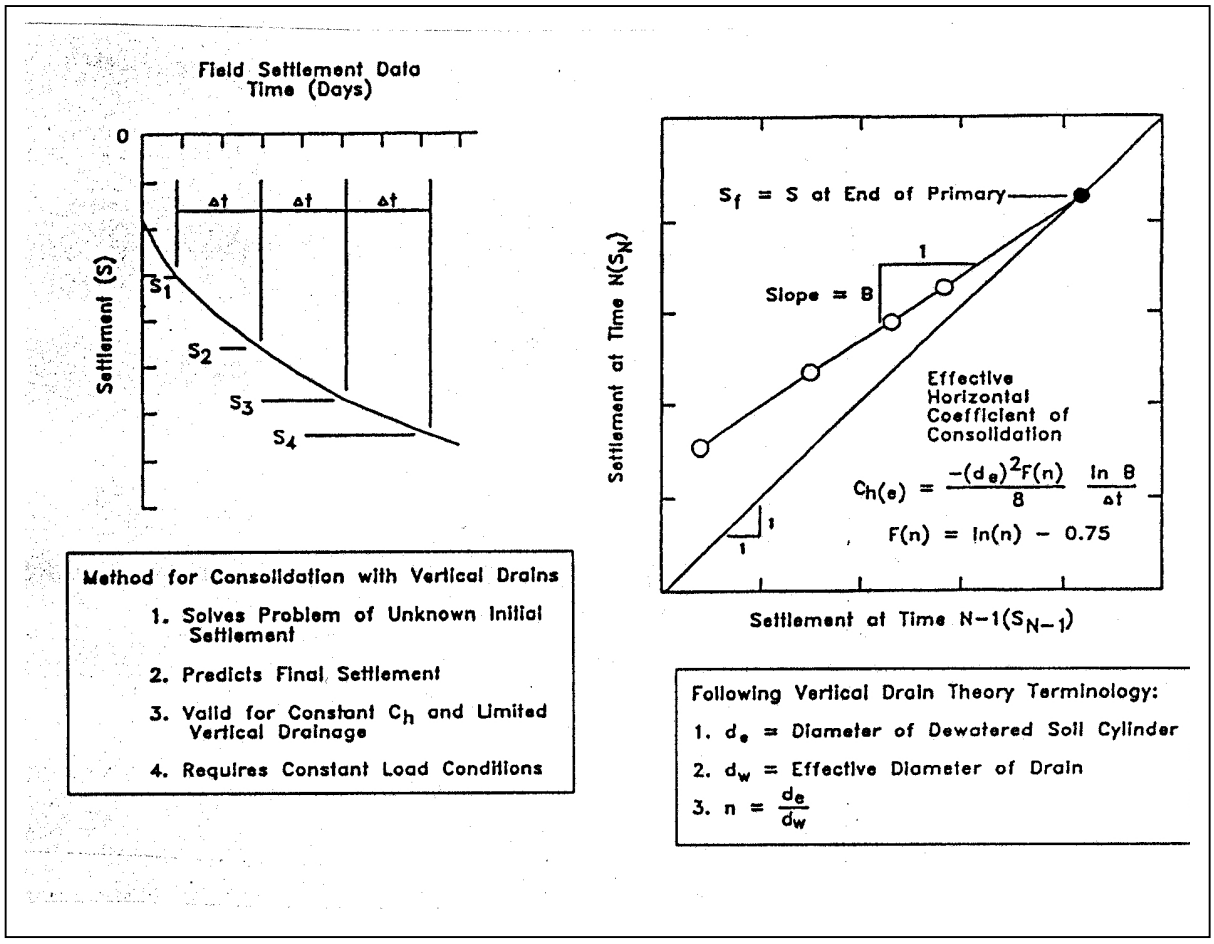


Figure 8. Asaoka Method for Determining c_h and End of Primary Consolidation.[8]

- **d**. There have been varying methods and recommendations to determine the equivalent circular drain size for a PVD. Various diameters ranging from 40 mm to 140 mm (1.6 in. to 5.5 in.) have been used, but the most common is to use 60 mm (2.4 in.). In fact, there would be very little difference in design if 50 mm or 100 mm (2.0 in. to 3.9 in.) were used.

- $\overline{U}_{\mathbf{h}}$. Normally, an average degree of consolidation desired is 90 to 95 percent 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**. The diameter of the cylinder of influence of each PVD. When using an equilateral triangular pattern, D is 1.05 times the spacing between each drain. In a square pattern, D 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 1-1.5 m (3-5 ft.) in homogenous clays; 1.2-1.8 m (4-6 ft.) in silty clays; and 1.5-2 m (5-6.5 ft.) in coarser soils.

- **t**. The time to achieve the required amount of consolidation ($\overline{U}_{\mathrm{h}}$) for a spacing D and drain diameter d.

There are three basic variables that can be manipulated in order to achieve a desired result from Equation 1. These variables are time, PVD spacing, and surcharge. If the surcharge is increased, yielding greater settlement, the PVD spacing can be increased in order to achieve the same amount of consolidation in a given time. [13]

Another approach is to add surcharge in order to decrease the time required for consolidation 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.

4.2 PRELIMINARY 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 can be estimated from liquid limit values using the graph shown in Figure 9.

Assuming that the desired percentage of primary consolidation has been established or is in the 90-95 percent range, the designer can then either use computer programs available for vertical drain design, or solve Equation 1 to determine a preliminary spacing consistent with available time. A new computer program FoSSA (Foundation Stress & Settlement Analysis) has been developed by FHWA to perform design procedures including those requiring stage construction.[14] Once the preliminary spacing is determined, the total project quantities can be computed from the project geometry.

4.3 PRELIMINARY EVALUATION EXAMPLE

A highway embankment is planned at a location where available borings indicate a profile of 6 m (19.7 ft.) of normally consolidated clay with sand lenses over rock. Total settlement, based on one-dimensional drainage is estimated at 300 mm (11.8 in.), 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.

Since the available construction time is on the order of 18 months, 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 1 year, without a surcharge load. To estimate feasibility and cost, a potential spacing must be determined.

From Figure 9, a coefficient of consolidation (c_v) of 0.001 cm^2/sec 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 1 can be solved for various drain spacing, using an equivalent diameter of drain of 60 mm (2.4 in.) to obtain the field time required to attain 90 percent consolidation. The solution indicates required times on the order of 300 days at an approximate spacing of 2.5 m (8 ft.) or 470 days, for an approximate spacing of 3 m (10 ft.).

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.

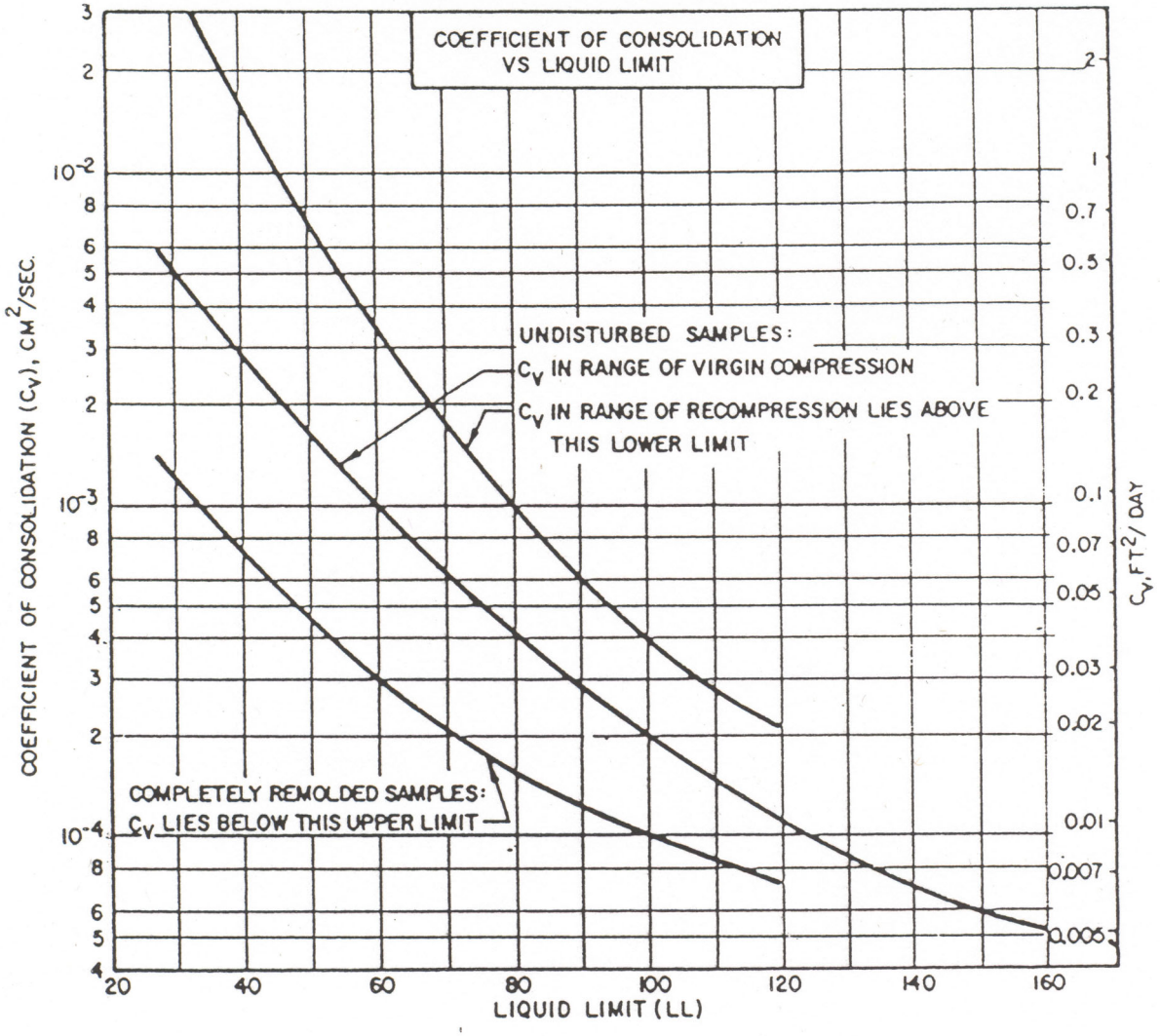


Figure 9. Coefficient of Consolidation vs. Liquid Limit.[8]

CHAPTER 5
CONSTRUCTION SPECIFICATIONS AND CONSTRUCTION CONTROL

PVD construction equipment, methods and materials, are well established with a proven track record of many successfully completed projects in the last few 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 improvement technology.

5.1 SPECIFICATIONS

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.

A general guideline specification follows. This specification is general in nature and should be modified for each project based on site conditions and performance requirements. A more detailed discussion on specifications, terms, and commentary is contained in FHWA RD-86-168.[(1)]

GUIDELINE SPECIFICATIONS FOR PREFABRICATED VERTICAL DRAINS

a. Description

The Contractor shall furnish all necessary plant, labor, and equipment and perform all operations necessary and incidental to the installation of PVDs in accordance with the contract drawings and as specified herein. The PVDs shall be the band-shaped type and spaced and installed as is shown on the plans or as directed by the Engineer.

b. Material

The PVD shall be a prefabricated plastic core wrapped in a filter of non-woven geotextile material. The core shall be fabricated with suitable drainage channels/studs.

The Contractor shall submit PVD samples and indicate the source of the proposed materials prior to delivery to the site and shall allow sufficient time (4 weeks) for the Engineer to evaluate the material. The material shall be one of the following products:

1. Alidrain
2. Alidrain S
3. Flexi-Drain
4. Amerdrain (Type 407)
5. Amerdrain (Type 417)
6. Mebra-Drain (7407)
7. Mebra-Drain (MD 88)
8. Colbonddrain CX 1000

c. Construction Details

Equipment

PVDs shall be installed with equipment that will cause a minimum disturbance of the subsoil during the installation operation. The PVDs shall be installed using a mandrel or sleeve that will be advanced through the drainage blanket and compressible soils to the required depth, using either static and/or vibratory methods. The mandrel shall protect the PVD material from tears, cuts, and abrasions during installation and shall be withdrawn after the installation of the drain. The mandrel shall be provided with an "anchor" plate or rod at the bottom to prevent soil from entering the bottom of the mandrel during drain installation and to anchor the bottom of the drain at the required depth when the mandrel is removed. The mandrel shall have a maximum cross sectional area of 80 square centimeters (12.4 square inches).

Installation Procedures

A minimum of four weeks prior to the installation of PVDs, the Contractor shall submit details of the sequence and method of installation to the Engineer for review and approval.

The submittal shall, at a minimum, contain the following specific information:

1. Size, type, weight, maximum pushing force, vibratory energy, and configuration of the

installation rig.
2. Dimensions and length of mandrel.
3. Details of drain anchorage.
4. Detailed description of proposed installation.
5. Proposed method(s) for addressing obstructions.
6. Proposed method(s) for splicing drains.

Approval by the Engineer will not relieve the Contractor of his responsibility to install PVDs in accordance with the plans and specifications. If, at any time, the Engineer considers that the method of installation does not produce a satisfactory drain, the Contractor shall alter his method and/or equipment as necessary to comply with the plans and specifications. Prior to the installation of PVDs within the designated areas, the Contractor shall demonstrate that his equipment, method, and materials produce a satisfactory installation in accordance with these specifications. For this purpose, the Contractor will be required to install trial PVDs at locations (number and pattern) designated by the Engineer. Payment for trial drains will be at the bid price per linear foot for the PVDs.

Construction Requirements

PVDs will be located, numbered, and staked out by the Contractor. The location of the PVDs shall not vary by more than 150 mm (1.5 in.) from the locations indicated on the drawings or as directed by the Engineer. The equipment shall be carefully checked for plumbness prior to advancing each wick, and must not deviate more than 50 mm per meter (0.6 in. per foot) from the vertical. The Engineer will reject wicks that are out of tolerance or that are damaged in construction, or that are improperly completed, and no compensation will be allowed for any materials furnished or for any work performed on such PVDs.

PVDs shall be installed from the working surface to the depth shown on the drawings or as directed by the Engineer. The Contractor shall use all possible means to attain the design depth of the drains.

After the installation of each drain, the PVD shall be cut so that a minimum of 150 mm (6 in.) of drain material extends above the top of the working surface. If the drainage blanket is fully in place, the PVD may be cut off at the ground surface.

The Contractor shall be permitted to use augering or other methods to loosen stiff or dense upper soils prior to the installation of the PVDs, provided that such augering or other method does not extend more than 600 mm (2 ft.) into the underlying compressible soils. If pre-augering is determined to be necessary, then the Contractor shall be permitted to accomplish the pre-augering prior to the installation of the drainage blanket. If the pre-augering is

performed prior to placement of the drainage blanket, a minimum length of 300 mm (12 in.) of PVD material must extend above the working surface in order to provide continuous drainage into the granular blanket.

If obstructions are encountered that cannot be penetrated by the drain installation equipment, including pre-augering as specified above, the Contractor shall notify the Engineer and under his inspection shall attempt to install an alternative drain within a 600 mm (2 ft.) radius of the original design location. A maximum of 2 attempts shall be made as directed by the Engineer.

The Engineer may vary the depths, spacing, or the number of PVDs to be installed, and may revise the plan limits for this work.

During the installation of the PVD, the Contractor shall provide suitable means of determining the depth of the drain at any given time.

Splices or connections in the PVD material shall be done in a workmanlike manner and so as to ensure continuity of the PVD material. A minimum overlap of 150 mm (6 in.) shall be required for each splice.

The Contractor shall take all necessary precautions for the protection of instrumentation devices. After instrumentation devices have been installed, the Contractor shall replace at his cost any instrumentation that is damaged or becomes unreliable as a result of his operation. The Contractor shall stop all PVD installation work until the damaged instrumentation is repaired.

d. Method of Measurement

PVDs will be measured by the linear meter (linear foot) for the full length of drainage wick complete and in place. In case of obstructions, the Contractor will be paid at the contract unit price for the number of linear meters (linear feet) measured from the theoretical top of the working platform to the elevation at which the obstruction was encountered.

e. Basis of Payment

Payment for PVDs will be made at the unit price per linear meter (linear foot), which price shall be full compensation for the cost of furnishing the full length of PVD material, installing the PVD, altering the equipment and methods of installation in order to produce the required end result in accordance with the plans and specifications, and shall also include the cost of furnishing all tools, materials, labor, equipment, and all other costs necessary to complete the required work. No payment will be made for unacceptable PVDs or for any

delays or expenses incurred through changes necessitated by improper or unacceptable material or equipment, but the costs of such shall be included in the unit price bid for this work.

Payment for the installation of PVDs encountering an obstruction shall be made for the full depth of PVD installed. Payment for obstruction removal shall be made at a time and material price.

Note: Where pre-augering of overburden is required, a separate pay item for the pre-augered length should be included. Alternately, pre-augering through dense or stiff soils located above the compressible layer can be made incidental and not a separate pay item.

5.2 INSPECTION GUIDELINES

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.

This section outlines construction monitoring procedures that should be considered for any PVD project. 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.

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.

a. 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 project, a contractor installed a 1.5-meter-high (5 ft.) haul road over a 600-mm (2 ft.) 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.

b. 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.

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.

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.

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 personnels' 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.

5.3 MONITORING AND 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 failure. The pore pressure devices (piezometers) are used for both calculation of achieved consolidation rate and 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.[3] 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.(5) 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 10. 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. Figures 11 and 12 show cumulative and incremental horizontal displacements at the location where the displacement ratios in Figure 10 have been computed.

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 on Figure 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.(3)

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 1.

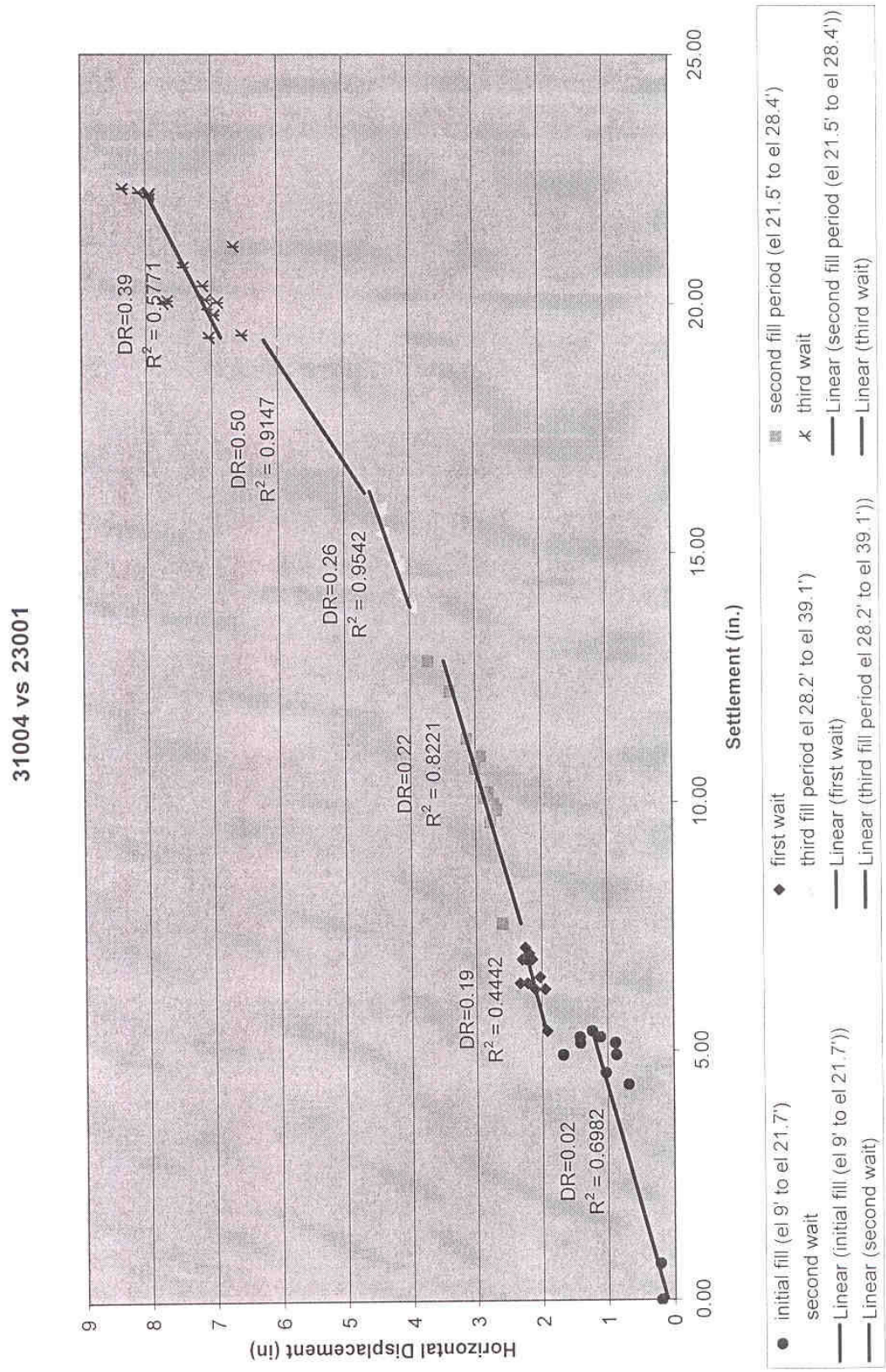


Figure 10. Displacement Ratio vs. Time and Stage Filling.

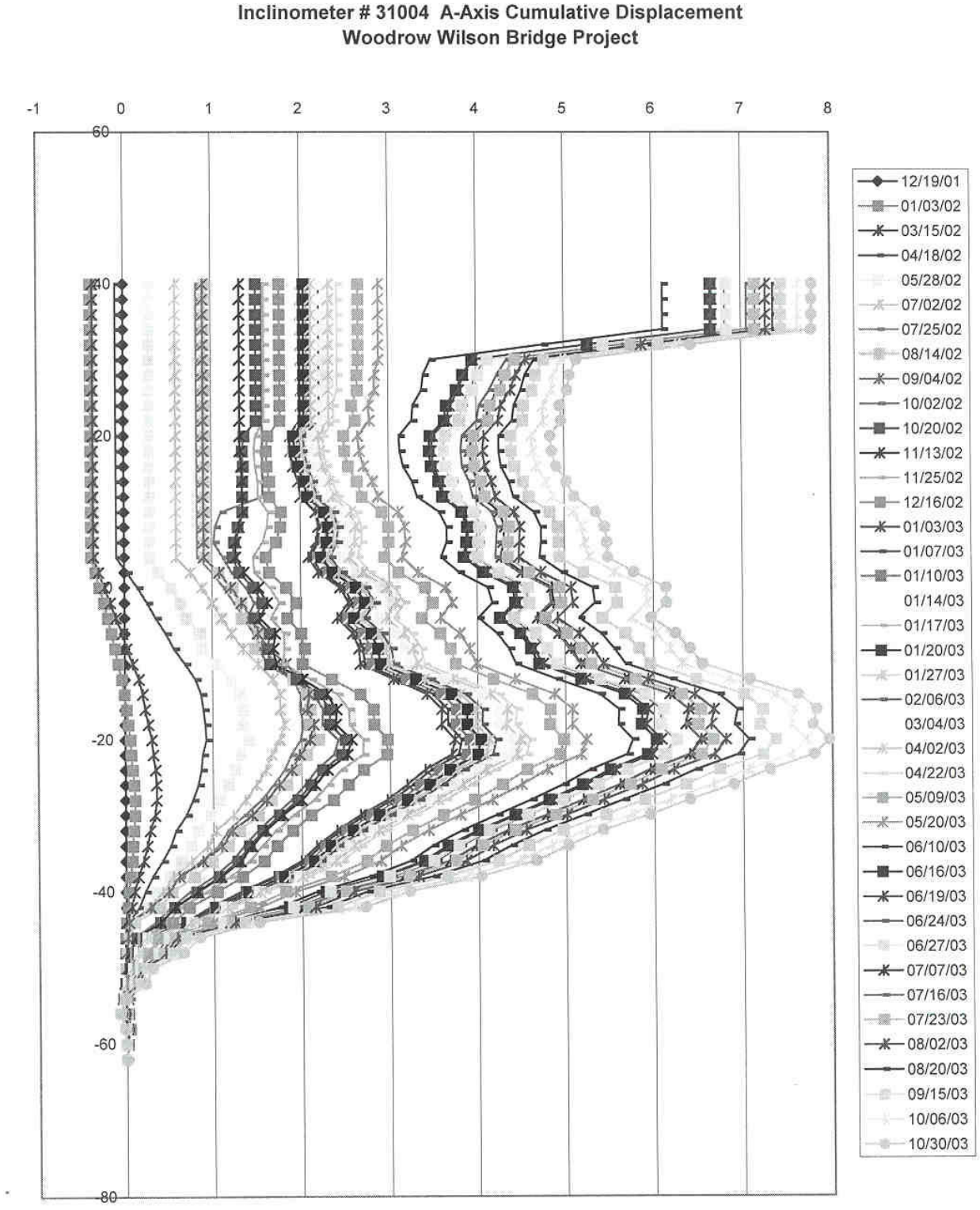


Figure 11. Cumulative Displacement vs. Depth.

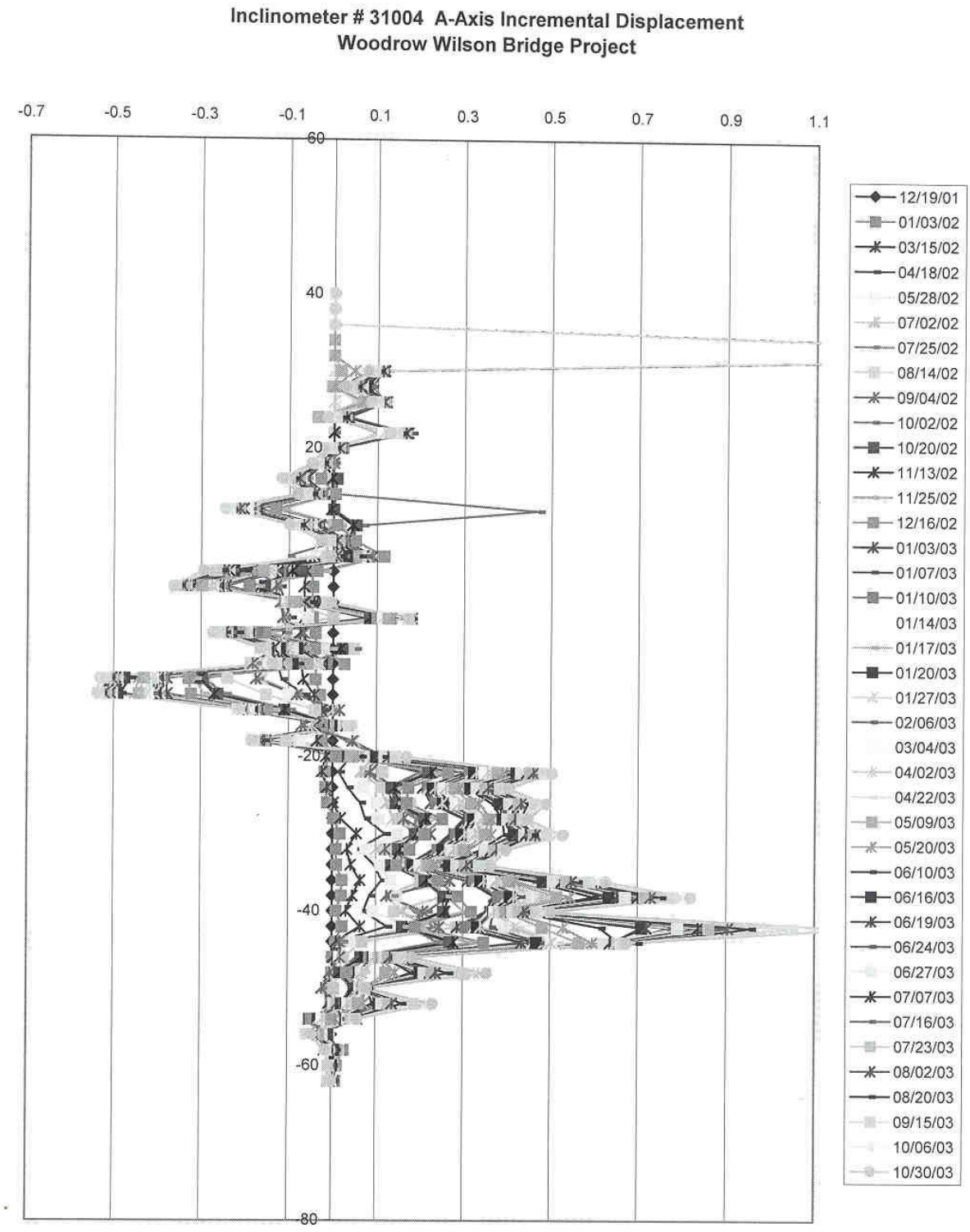


Figure 12. Incremental Displacement vs. Depth.

CHAPTER 6
COST DATA

6.1 COST COMPONENTS

Often when reporting the cost of PVDs, only the actual cost of the installed PVDs is considered, rather than the cost of other ancillary items. Table 3 details other factors affecting the total cost of the PVD solution. 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.

Table 3. Factors affecting the total cost of a prefabricated vertical drain solution.

• 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

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. It is also difficult to anchor PVDs in extremely soft soils.

- *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 both consolidation time, time for installation of the PVDs, and time for construction of the embankment and/or surcharge. Added 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. The more surcharge that can be placed, the faster the 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 and gravel 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 stage construction is required.

6.2 SUMMARY OF TYPICAL PROJECTS

Typical cost of PVD installations can be divided into three categories: small (3,000-10,000 m) (10,000-33,000 ft.), medium (10,000-50,000 m) (33,000-164,000 ft), and large (50,000 m and greater) (164,000 ft. and greater).

The following are typical unit price ranges for projects where the soils do not present major difficulty in penetration, do not require special equipment or are not at unusually difficult sites:

Site Category	Unit Price Range	(Unit Price Range)
Small	$2.25 to $4.00 per m	($0.70 to $1.25 per ft)
Medium	$1.60 to $2.50 per m	($0.50 to $0.75 per ft)
Large	$0.90 to $1.60 per m	($0.30 to $0.50 per ft)

Usually added to these costs is a mobilization charge of $8,000 to $10,000, as well as the cost of the drainage blanket and instrumentation. Where there are severe conditions in the area of weather, labor conditions, site conditions, and/or difficulties in penetration, the unit cost could be significantly higher.

No commercial vacuum consolidation project has been constructed in the United States to date. Present estimates for implementing such a project range from $ 70 to $100 per square meter of surface area ($7 to $10 per square foot).[12]

CHAPTER 7
CASE HISTORIES

The following case histories illustrate successful projects.

7.1 AIRPORT RUNWAY AND TAXIWAY EXTENSION

Project Description

The Quad City Airport located in Moline, Illinois, required the extension of one of its runways by 300 meters (1,000 ft.) to meet future needs and provide a secondary runway for larger aircraft. Because an existing highway limited any extension at one end, the runway had to be extended toward a bluff at the other end. In order to maintain the proper flight path clearance, the elevation of the end of the runway would require as much as a 4.5-meter (15 ft.) fill over existing soft alluvial soils. The width of the runway was 45 meters (150 ft.).

Subsurface Conditions

The subsoils consisted of silty clay overlying fine sand of medium density. The upper 1.5-2.0 meters (3.3-6.6 ft.) of the silty clay was medium stiff with "N" values ranging from 5-10 blows per 300 mm (1 ft.). This material was slightly desiccated and had moisture contents ranging from 22 percent to 30 percent. The lower portion of the silty clay layer was soft to very soft with "N" values from 1-4 blows per 300 mm (1 ft.) and a moisture content ranging from 30-45 percent. The total depth of the silty clay layer ranged from 4.5-7.6 meters (15-25 ft.).

Design Concerns

The designers were very concerned with post-construction settlement resulting from the added fill, in order to maintain the proper grade for the runway. Settlement projections varying from 120 mm (4.7 in.) near the end of the existing runway to in excess of 300 mm (12 in.) at the greatest fill height could not be tolerated. Various solutions, such as removal and replacement, surcharging, and PVDs, were investigated. The high cost of removal and replacement eliminated this option. The time rate of settlement eliminated the option of just building the embankment and waiting for settlement to occur. Surcharging would not only be costly because of the need for removal and disposal after completion of settlement, but it would also create a slope stability problem. The design also called for a twin set of 1.80-meter (6-ft.) diameter culverts to be extended under both the runway and taxiway. While these culverts could be designed for the total amount of settlement anticipated, it was necessary that they be at the final elevations prior to the opening of the runway.

Therefore, the use of PVDs to accelerate the settlement, combined with a small amount of surcharge that could be used elsewhere on the site, was chosen as the solution.

Another concern to the designers was the availability and cost of a clean sand to be used for a drainage blanket. The problem was solved by designing a system of strip drains, utilizing a 300 mm x 25-mm (12 in. x 1 in.) strip drain, which was outletted into a gravel and pipe trench under the outer edge of the embankment. Each strip drain was connected to four rows of PVDs by extending the wick material over to the strip drain. This was the first use of horizontal strip drains in lieu of a drainage blanket.

Design

Results from both field and laboratory testing indicated that a maximum total settlement of 365 mm (14 in.) would occur under approximately 1.7 m (5.2 ft.) of surcharge and 4.5 m (15 ft.) of fill. Based on consolidation coefficients, it was determined that a 1.5-meter (5-ft.) triangular pattern spacing of the PVDs would result in a settlement time period of approximately 6 months, once full height was attained. The project was divided into two phases, since the taxiway wasn't as critical as the runway. The runway project was designed first, with a PVD quantity of approximately 77,000 linear meters (252,600 ft.). The area was extended to cover part of the taxiway so that the future project of the taxiway could be completed without any settlement effect to the runway.

Project Results

The PVD installation began in the middle of June, and just under 10,000 PVDs, averaging 7 meters (23 ft.) in depth, were installed in a 3-week time period. The horizontal drains were completed shortly thereafter. Actual settlements ranged from 100-400 mm (4-16 in.) in the worst area, and were completed in 3 to 4 months, within the design time of 6 months.

Since stability was not a concern, only settlement plates were necessary to determine the amount and time rate of consolidation.

Project Cost

This project was somewhat unique in that the PVDs were bid on a per drain basis instead of the more common unit of linear meter (ft.). The unique unit of measurement was selected because the PVDs were short, but had to be extended to the strip drains at the surface. Subcontractor prices for the PVDs were approximately $11.50 per installed drain, or slightly in excess of $1.30 per linear meter ($0.40 per ft.) with a mobilization charge of $10,000. Installed horizontal strip drain prices were $4.80 per linear meter ($1.50 per ft.).

7.2 APPROACH RAMPS FOR BRIDGE REPLACEMENT

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 90 meters (300 ft.) beyond the old abutment. The total alignment of the new embankment centerline and bridge would be approximately 21 meters (69 ft.) 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.

Subsurface Conditions

The generalized subsurface soil profile indicated a 2-to-5 meter (6.6-to-16.4 ft.) fill layer of cinders and slag over 0.75 meters (2.5 ft.) of peat (under the west embankment only), 10-14 meters (33-46 ft.) of soft silty clay, and finally, a 3-meter (10-ft.) layer of sand overlaying limestone bedrock. The ground water table was within 0.5-1.0 meters (1.6-3.3 ft.) of the existing surface.

Design Concerns

Construction of the new approach embankments, varying in height from 5-11 meters (16-36 ft.), would result in considerable settlements. The west approach embankment would result in settlements ranging from 600-900 mm (24-36 in), 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 300-600 mm (12-24 in.) were expected, but the time of the settlement would be similar to the west approach. The designers used PVDs 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 17 meters (56 ft.) with a headroom clearance of only 8 meters (26 ft.).

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 4.5 meters (15 ft.), 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 8.5 meters (28 ft.) without foundation treatment. However, after the strength gain as a result of PVD accelerated consolidation, embankments could be constructed to heights of 11 meters (36 ft.), as required.

Design

The west embankment was expected to settle up to a maximum of 900 mm (36 in.) under maximum fill height, and the east embankment a 600-mm (24-in.) maximum. A surcharge height of 900 mm (36 in.) above final grade elevation accelerated settlement. Where settlement was maximum and stability was of concern, the PVD spacing was 1.2 meters (4 ft.) in a triangular pattern. In areas of lesser settlement, where stability was not a concern, the pattern spacing was 2.1 meters (7 ft.).

The total quantity of the original estimate was 70,000 linear meters (229,600 lft.) of PVDs. Added to this quantity was the cost of 5,000 m^3 (6,500 yd^3) of a granular material for the drainage blanket and 16,000 m^2 (19,100 yd^2) of a geotextile. New York State DOT estimated the total cost of the PVD solution, including auxiliary items, to be in excess of $800,000.

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 25,000 m (82,000 ft.) 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 6 meters (20 ft.) long, with subsequent sections of 5 m (16 ft.). 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 33,500 m (110,000 ft.).

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

Project Results

Typical piezometers and settlement data from both the east side and west approach are shown in Figures 13 and 14. Total maximum settlement of the west approach was 700 mm (28 in.); on the east approach, the maximum settlement was 600 mm (24 in.). The settlement was achieved in both locations in approximately 4 to 5 months and agreed fairly well with predictions. While slope stability was not believed to be a concern, 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.

Project Costs

This project was somewhat unique in that the use of several different installation units had to be bid less than 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 $6.90 per m ($2.10 per ft.); (2) granular drainage blanket at $22.00 per cubic meter ($17.00 per cu. yd.); and (3) geotextile at $1.70 per square meter (1.40 per sq. yd.). 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 has 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.

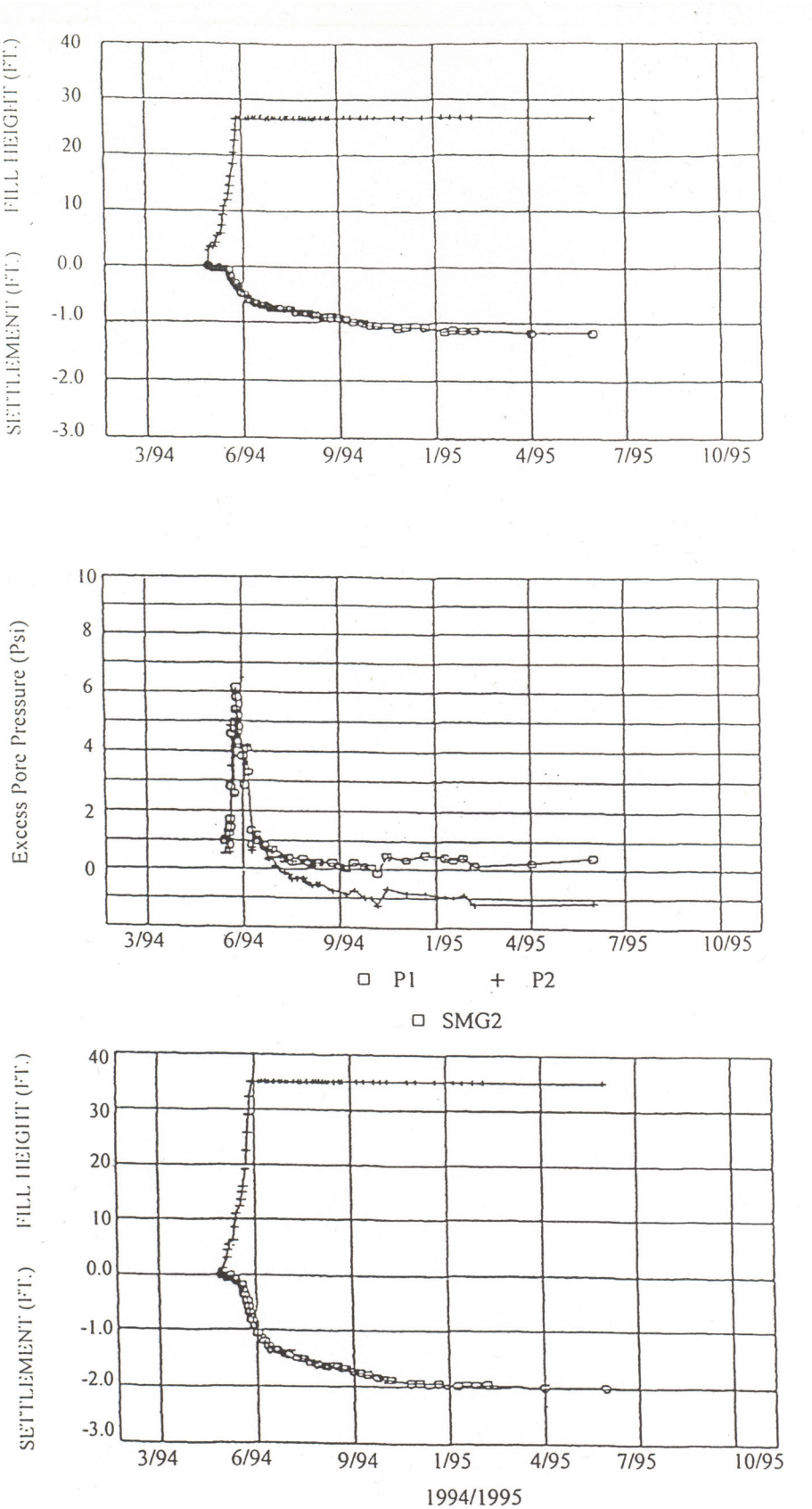


Figure 13. Tifft Street West Approach Embankment Field Data.

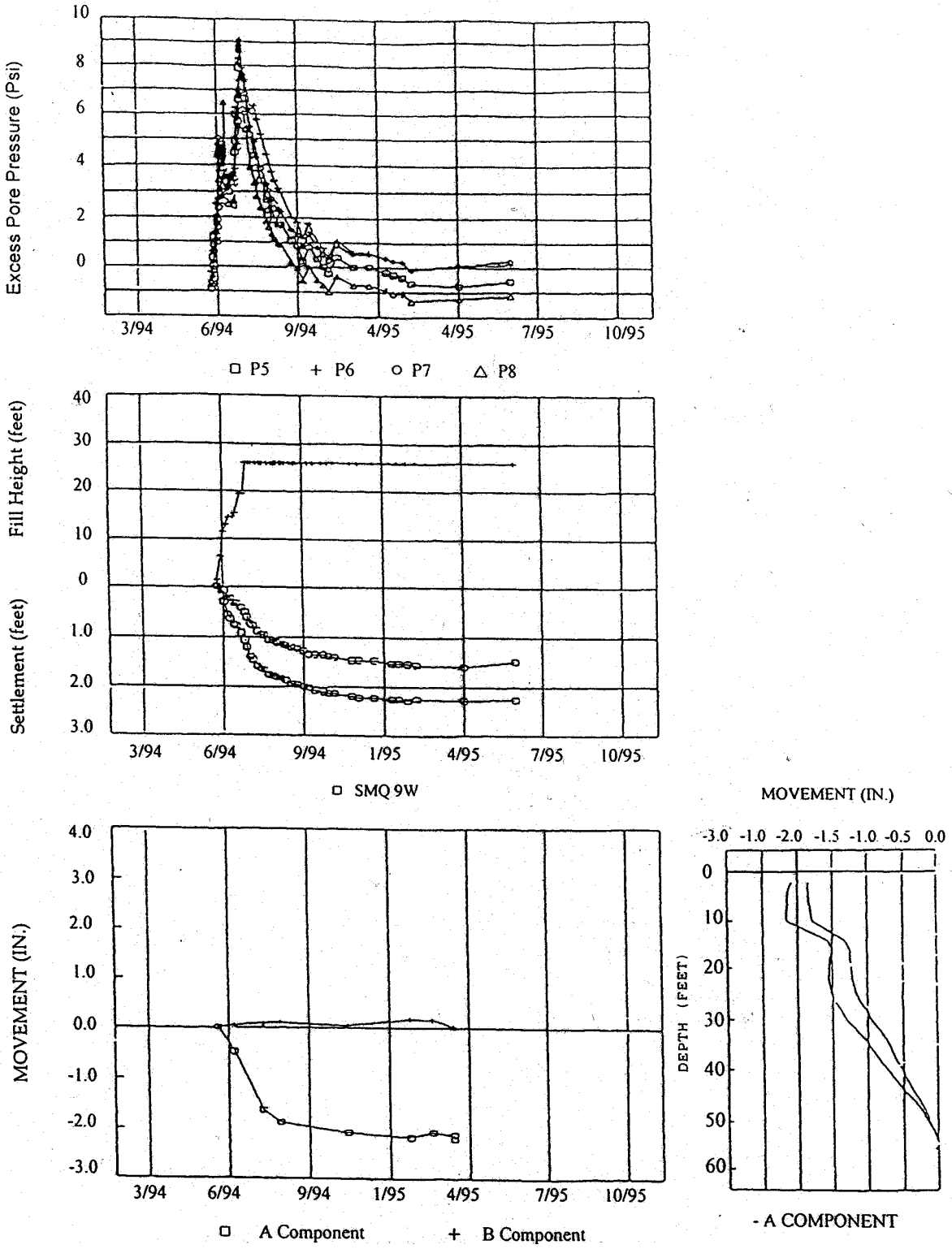


Figure14. Tifft Street East Approach Embankment Field Data.

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Technical Summary # 2:

LIGHTWEIGHT FILLS

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CHAPTER 1
DESCRIPTION AND HISTORY

1.1 HISTORICAL OVERVIEW

The compacted unit density of most soil deposits consisting of sands, silts, or clays ranges from about 18-22 kN/m^3 (115-140 lb/ft^3). On some projects, it is desirable to use a material with a lower density in order to reduce the magnitude of applied loads. The use of conventional earth fill material could result in significant settlement or decreased stability, such as a potential bearing capacity failure or a lower than desired factor of safety against a deep-seated failure. In this situation, the use of a lightweight fill material would likely result in reduced settlement and increased stability. Also, lightweight fill can be used to reduce the horizontal forces applied to earth retaining structures, and to increase an embankment's resistance to seismic loads (low unit weight (density) results in lower seismic inertial forces).

The worldwide interest and use of lightweight fill materials has led to the publication by the Permanent International Association of Road Congresses (PIARC) of an authoritative reference, *"Lightweight Filling Materials,"* in 1997.[1]

Many types of lightweight fill materials have been used for roadway embankment construction. Some of the more common lightweight fills are listed in Table 1. There is a wide range in density of the lightweight fill materials, but all have a density less than conventional soils. Chapter 3 discusses the composition and sources of the lightweight fill materials listed in Table 1.

Some lightweight fill materials have been used for decades, while others are relatively recent developments. Wood fiber has been used for many years by timber companies for roadways crossing peat bogs and low-lying land, as well as for repair of slide zones.[1]

The steel-making companies have produced slag by-product since the start of the iron- and steel-making industry. Initially, the slag were stockpiled as waste materials, but beginning around 1950, the slag were crushed, graded, and sold for fill materials.[2,3,4]

Geofoam is a generic term used to describe any foam material used in a geotechnical application. Geofoam includes expanded polystyrene (EPS), extruded polystyrene (XPS), and glassfoam (cellular glass). 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, the use of geofoam was extended as a lightweight fill for a project in Norway.[1,5]

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 U.S. Army Corps of Engineers used foamed concrete as a tunnel lining and annular fill. This product is generally job-produced as a cement/water slurry, with preformed foam blended for accurate control and immediate placement.

Shredded tires are a relatively recent source of lightweight fill materials. 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. In most locations, the tires are stockpiled, but they are unsightly and present a serious fire and health hazard. Shredded tires have been used for lightweight fill in the United States and in other countries since the mid 1980s. Over 85 fills using shredded tires as a lightweight fill have been constructed in the United States.[1]

Table 1. Densities and approximate costs for various lightweight fill materials.

Fill Type	Range in Density kg/m^3	Range in Specific Gravity	Approximate Cost[1] $/m^3
Geofoam (EPS)	12 to 32	0.01 to .03	40 to 85[2]
Foamed Concrete	320 to 970	0.3 to 0.8	40 to 55
Wood Fiber	550 to 960	0.6 to 1.0	12 to 20[2]
Shredded Tires	600 to 900	0.6 to 0.9	20 to 30[2]
Expanded Shale, Clay & Slate (ESCS)	600 to 1040	0.6 to 1.0	40 to 55
Fly Ash	1120 to 1440	1.1 to 1.4	15 to 21[3]
Boiler Slag	1000 to 1750	1.0 to 1.8	3 to 4[3]
Air-Cooled Slag	1100 to 1500	1.1 to 1.5	7.50 to 9[3]

[1] See Chapter 6 for details on cost data

[2] Price includes transportation and placement cost

[3] FOB plant

In 1995, three tire shred fills with a thickness greater than 8 m (26 ft.) experienced an unexpected internal heating reaction. As a result, FHWA issued an Interim Guideline to minimize internal heating of tire shred fills in 1997, limiting tire shred layers to 3 m (10 ft.).[8]

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 the Port of Albany, New York marine terminal, where lightweight fill was used behind a bulkhead to reduce the lateral pressures on the steel sheeting. Other projects include construction of roadways over soft ground. The existing high-density soils were partially removed and replaced with lightweight aggregate to reduce settlement. Other projects have included improvement of slope stability by reduction of the gravitational driving force of the soil in the slope and replacement with a lightweight fill. [1,9,10,11]

Waste products from coal burning include fly ash and boiler slag. Both of these materials have been used in roadway construction.[1] 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. In 1984, a project survey found that fly ash was used in the construction of 33 embankments and 31 area fills. Boiler slag has been used for backfill since the early 1970s. Many state highway department specifications allow the use of boiler slag as an acceptable fine or coarse aggregate.

1.2 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 also presents guidelines for preparation of specifications along with suggested construction control procedures. Four case histories are presented to demonstrate the effectiveness of lightweight fills for specific situations. Approximate costs for the various lightweight fill materials are also presented.

This technical summary should not be considered to be a design manual, 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.

CHAPTER 2
APPLICATIONS AND LIMITATIONS

2.1 APPLICATIONS

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, either the foundation soils must be improved or the embankment load reduced. Lightweight fills fulfill the latter function. A brief overview of lightweight fill use or function is summarized as follows:

a. 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:[12]

- 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
- the use of deep foundations to transfer some of the load through the weak foundation deposits
- the use of stabilizing berms adjacent to the embankment or flatter side slopes
- the 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 Chapter 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.

b. Shear Strength

Granular lightweight fills have an angle of shearing resistance similar to natural soils, while cemented 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.

c. Compressibility

Many lightweight fill materials, such as foamed 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 1 m (3 ft.) 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.

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.

d. 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 foamed concrete or 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.

e. 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

- 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.

f. 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 geofoam, ESCS, and low-density cellular (foamed) concrete, have been successfully installed in bad weather. The first case history of Chapter 7 describes one project where this was accomplished.

g. 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.[13] The lower unit weight of the material results in lower inertial forces under seismic loading.

2.2 LIMITATIONS/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. Shredded tires are very resilient when placed, thereby requiring somewhat unconventional compaction procedures. Foamed 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.,* geofoam) must be protected to ensure longevity. Because geofoam is subject to deterioration from hydrocarbon spills, a concrete slab or geomembranes are 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.

- *Environmental concerns*. Some of the lightweight fill materials generate leachate as water passes through these deposits. Fortunately, design methods 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 (500 mm (20 in.), minimum) of soil and/or paving materials over the surface of the lightweight fill material.

CHAPTER 3
LIGHTWEIGHT FILL MATERIALS

3.1 INTRODUCTION

Lightweight fill materials include both recycled and manufactured materials. 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 land filled 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 20 years.

Manufactured materials consist of products such as 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.

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

3.2 GRANULAR LIGHTWEIGHT FILLS

a. 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 References 17 and 19. Figure 1 illustrates the placement of wood fiber in embankment construction.

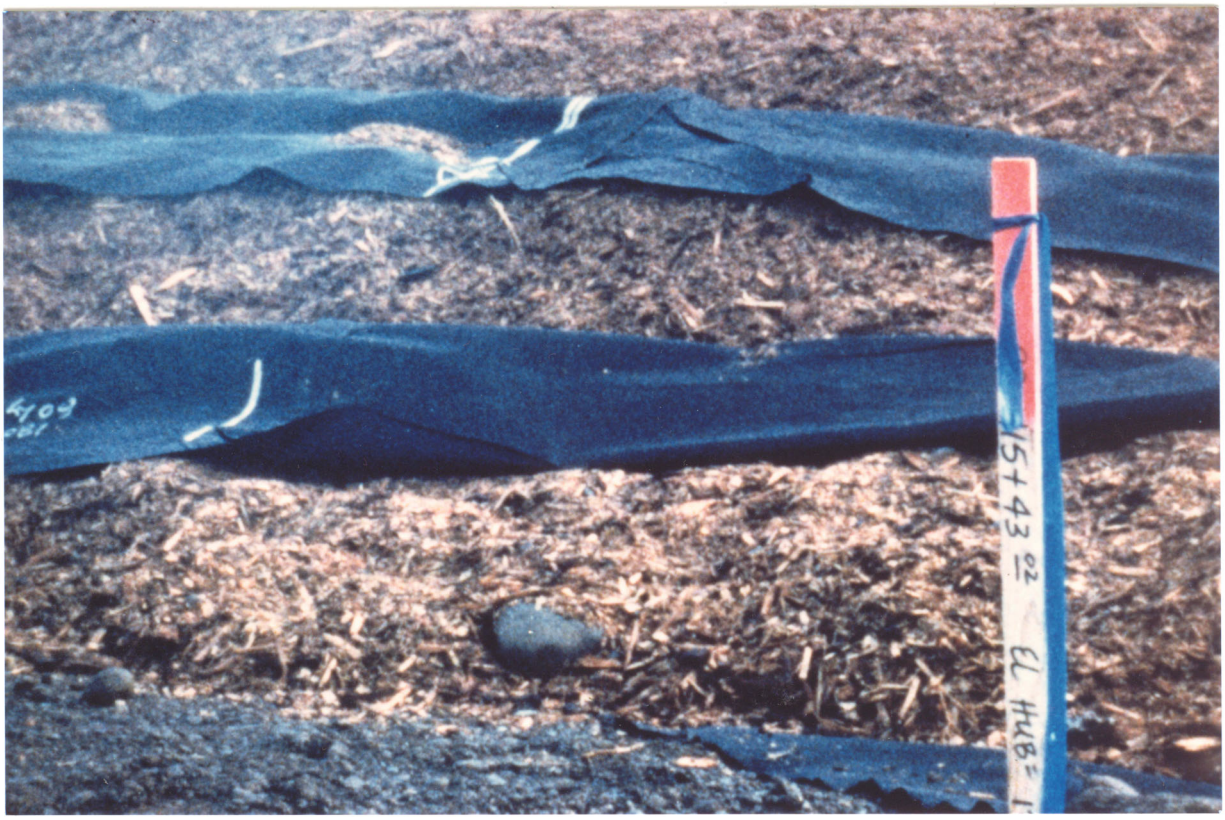


Figure 1. Embankment Construction with Wood Fiber.

b. 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.[14] 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.

c. 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 C 618 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 C 618. This class of fly ash has pozzolanic properties.

In recent years, the primary use for type C and F fly ash 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 foamed concrete.

d. Boiler Slag

Boiler slag is produced in power plants from wet bottom and cyclone 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 6-9 mm (⅛-⅜-in.) in size. It is generally composed of silica and ferric oxide particles of angular and irregular shape.

e. 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. 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.

Figure 2 illustrates the placement of expanded clay lightweight fill during the early stages of embankment construction.

f. Shredded Tires

Shredded tires are produced by mechanically cutting tires into chips that are generally in the size range of 100-200 mm (4-8 in.). These chips are durable, coarse grained, free draining, and have a low compacted density. Each cubic meter of tire chip fill contains about 100 waste tires, so there is a potential for using a large number of tires in highway construction. Both steel- and glass-belted tires are generally used to produce shredded tires.

Figure 2. Placing Expanded Clay Fill.

The engineering properties of shredded tires have been reported in technical publications and have been summarized by Humphrey.[22] After some instances of spontaneous combustion of tire shred fills, FHWA has issued recommendations for this type of construction to preclude further spontaneous combustion.

ASTM D6270 "Standard Practice for Use of Scrap Tires in Civil Engineering Applications" summarizes certain uses, and details material characterization, construction practices, and levels of potential leachates.

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 other locations, the product is used as a fuel source. Thus, the economics will depend upon the location.

Figure 3 illustrates fill construction with tire shreds.

Figure 3. Fill Construction with Tire Shreds.

3.3 LIGHTWEIGHT FILL MATERIALS WITH COMPRESSIVE STRENGTH

a. 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. The final product is generally a block similar to a large brick, but with typical dimensions of 2 m (6.6 ft.) long, 1 m (3.3 ft.) wide and 0.5 m (1.6 ft.) thick. Different block size can be made depending upon the size of the mold. Blocks can be factory- or field-cut to any size or shape.

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. It is possible to incorporate additives into the manufacturing process so that the blocks are flame retardant and insect infestation resistant.

Figure 4 illustrates embankment construction with geofoam.

NCHRP Project 24-11 contains design guidelines for the use of EPS geofoam in roadway embankments and bridge approaches. The document contains design guidelines or procedures, as well as appropriate material and construction standards.[6]

b. Foamed Concrete

Low-density cellular concrete is produced by introducing a preformed foam (similar in appearance to shaving cream) into a 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 because the mixing action of a ready mix truck does not produce an acceptable slurry. The mixture may include some sand.

After these materials, (cement, water, preformed foam), are blended to the specified density, the resulting slurry is pumped into place. This slurry has the consistency of a malted milk with the same water/cement ratio as regular concrete. The low-density cellular concrete is cast in lifts ranging from 0.3 to 1.2 m (1 to 4 ft.) or more depending upon the specific application. Since it is cementitious in nature, hydration solidifies it. It requires neither compaction nor protection.

The quality of low-density cellular concrete is monitored through its 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.

Figure 5 illustrates the placement method.

Figure 4. Embankment Construction with Geofoam.

Courtesy of Aning Johnson Co.

Figure 5. Foamed Concrete being Placed for Embankment in Illinois.

CHAPTER 4
DESIGN CONCEPTS

4.1 INTRODUCTION

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 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.[12,15] If settlement is a concern, the reduced density of the fill should be used in the settlement calculations. For fills characterized by compressive strength (geofoam, foamed concrete), some unique design criteria must be evaluated.

- An evaluation of the durability, water absorption potential, corrosion 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 shown in Table 1 and are discussed in Chapter 7. However, the cost will vary for each particular site depending upon the availability of the lightweight fill materials at the project site.

4.2 DESIGN AND CONSTRUCTION GUIDELINES

As a guide for preliminary planning and design, Tables 2 through 12 summarize some of the important geotechnical engineering parameters for lightweight fill materials. In addition, the tables also list environmental, construction, and design considerations. 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.

a. Granular Lightweight Fills

Tables 2 through 7 summarize the important design and construction criteria associated with granular lightweight fills. Design proceeds using the indicated variables and conventional geotechnical methods associated with granular soils.

Table 2. Wood fiber design and construction guidelines.

Design Parameters	
Moist Density:	720-960 kg/m^3 (45-60 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 = 9-10 MPa in top 0.6 m (2 ft.) roughly corresponding to a CBR of 1.
Environmental Considerations	
Potential environmental effects of the leachate include • depletion of available dissolved oxygen in ground water. • lowering of ground water pH because of acidic nature of leachate, which has pH of 4-6. • 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 150 mm (5.9 in.) maximum to prevent development of large voids. Less than 30 percent should be finer than 12 mm (0.5 in.) 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 0.6 m (2 ft.) or more to protect slope from erosion and minimize deterioration of wood fibers. Restrict height of fill to 5 m (16.4 ft.) and reduce air penetration into wood to minimize the possibility of spontaneous combustion.	
Construction Considerations	
Truck-mounted equipment is used to spread fiber in 0.3-0.5 m (12-20 in.) lifts. Two passes with a fully loaded hauling truck weighing 15 Mg (33 kips) or more is usually sufficient to properly compact wood fiber.	

Table 3. Air-cooled blast furnace slag design and construction guidelines.

Design Parameters	
Compacted Moist Density:	1120-1500 kg/m^3 (70-94 pcf) varies with size and gradation.
Gradation:	Can be graded to any specified size from 100 mm (3.9 in.) 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 400 mm (15.7 in.). The amount passing .074 mm (#200) should be limited to 5-7 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 1-6 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 4. Fly ash design and construction guidelines.

Design Parameters	
Density Range - Compacted	1120-1440 kg/m^3 (70-90 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:	Cc = 0.05 to 0.37 Ccr = 0.006 to 0.04
Grain Size Range:	0.005-0.074 mm
Specific Gravity:	1.9-2.5
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 has declared fly ash as non-hazardous.(16)	
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 0.6 m (2 ft.) 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: 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 250 mm (9.8 in.) lifts and compact the fly ash immediately after spreading. The use of test strips to develop the most efficient compaction procedures is advisable.	

Table 5. Boiler slag design and construction guidelines.

Design Parameters	
Dry Density, Loose:	960-1250 kg/m^3 (60-78 pcf)
Dry Density, Compacted:	1440-1750 kg/m^3 (90-109 pcf)
Angle of Shearing Resistance:	38° to 42°
Coefficient of Permeability:	0.3-0.9 mm/s
Grain Size Range:	0.05-10 mm
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 0.6 m (2 ft.) 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.	

Table 6. Expanded shale, clay, and slate (ESCS) design and construction guidelines.

Design Parameters	
Dry Density, Compacted:	800-1040 kg/m^3 (50-65 pcf)
Dry Density, Loose:	640-860 kg/m^3 (40-54 pcf)
Angle of Shearing Resistance:	loose 35°, compacted 37° to 44°
Grain Size Gradation:	5-25 mm
Permeability:	High
Coefficient of Subgrade Reaction:	loose: 9-10 MN/m^3 (33-37 pci) compacted: 38-42 MN/m^3 (140-155 pci)
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 0.8 m (2.6 ft.) 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 1 m (3.3 ft.) or less. Fill should be unloaded at side of fill area, then distributed with lightweight equipment with a contact pressure of 30 kN/m^2 (4.4 psi) or less. Field density may be approximated in the laboratory by conducting a modified one-point AASHTO T-272 density test.	

Table 7. Shredded tire design and construction guidelines.

Design Parameters

Dry Density	250-530 kg/m^3 (15.6-33 pcf) loose
(Depends on size of pieces)	720-900 kg/m^3 (45-56 pcf) compacted
Angle of Shearing Resistance:	19° to 25°
Cohesion Intercept:	8-11 kPa (165-230 psf). Use 0 for design.
Compressibility:	Strain of 10 percent over a range of 50-380 kPa (1000-8000 psf) vertical stress.
Permeability:	5-35mm/sec
Gradation:	100-200 mm
Coefficient of Lateral Earth Pressure:	Varies from 0.26 to 0.47. Density dependent.

Environmental Considerations

The Minnesota Pollution Control agency studied leachate from waste tire samples. Their findings indicate the following:

- Tire samples exposed to acidic conditions leach higher concentrations of metals than those subjected to neutral or basic solutions.
- The metals that leached included barium, cadmium, chromium, lead, selenium, and zinc.
- In neutral solutions (pH=7.0), tire samples did not leach any detrimental contaminants.
- Soil samples taken from shredded-tire field sites displayed constituent concentrations comparable to those found in natural settings.

Combustion Potential

FHWA Interim Guidelines to minimize internal heating of tire shred fills.

- Class I fills < 1 m (3.3 ft.) thick
  Maximum of 50 percent passing 38 mm (1.5 in.) sieve.
  Maximum of 5 percent passing 4.75 mm (#4) sieve.
- Class II fills (1-3 m (3.3-10 ft.) thick)
  Maximum of 25 percent passing 38 mm (1.5 in.) sieve.
  Maximum of 1 percent passing 4.75 mm (#4) sieve.
  Less than 1 percent metal fragments not encased in rubber.
  Infiltration of water and air into tire shred fill shall be minimized.
  Tire chips should be separated from the surrounding soil with a geotextile.
  Drainage features that could provide free access to air should be avoided at the bottom of the fill.

Table 7 (cont.)

Design Considerations Keep the shredded-tire fill above the water table. Limit layers to 3 m (10 ft.) in thickness. Provide good surface drainage of roadway surface to avoid water seepage through the shredded-tire fill. Use geotextiles above and below tire fill to prevent migration of surrounding soils into the fill. Limit maximum size of tire chip to 600 mm (24 in.) in length to prevent development of large voids. No crumb rubber is permitted. 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. At least one sidewall must be severed from the face of the tire. Place a minimum 0.9-m (3-ft.) thick soil cap on the top and side slopes of the tire chip fill to minimize pavement deflections and provide confinement.
Construction Considerations Spread using a track-mounted dozer in a lift thickness of 0.9 m (3 ft.) 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.

b. Fills with Compressive Strength

Geofoam

The most comprehensive design, material, and construction guidelines on the use of EPS geofoam for highway construction has been summarized in NCHRP 24-11.[6] Additional design information is summarized by Horvath.[5]

The overall design process when using EPS geofoam is divided into three phases in order to consider the interaction between the three major components in the embankment.[6]

- 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.
- Design for internal stability within the embankment mass. The design must ensure that the geofoam mass can support the overlying pavement system without immediate and time dependent creep compression.
- Design of an appropriate pavement system for the subgrade provided by the underlying geofoam blocks.

External stability analyses generally follow traditional geotechnical procedures, although stress distribution must consider a non-homogeneous embankment.[6] Stability analyses require the modeling of undrained shear strength for the geofoam, which presents some uncertainties. For shear strength, NCHRP 24-11 recommends the use of one-quarter of the compressive strength of the geofoam.

Internal stability analyses primarily consist of selecting an EPS geofoam type with adequate properties to support the overlying pavement system and the traffic load without excessive settlement of the surface. The current design approach is a deformation-based methodology using the total stress from all loads on the EPS blocks and elastic limit stress and the initial tangent modulus to evaluate load-induced deformations. Table 8 provides the minimum recommended values of elastic limit stress for various EPS densities.

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. Table 9 provides a summary of EPS engineering parameters typically used in pavement design.[6]

Note that NCHRP 24-11 provides detailed design methods, examples, typical construction details, and design charts for external, internal, and pavement design.

Table 10 summarizes the range of design parameters and construction criteria associated with the use of Expanded Polystyrene (EPS) geofoam.

Table 8. Minimum allowable values of elastic limit stress and initial Young's modulus for the NCHRP 24-11 EPS material designations.

Material Designation	ASTM	Dry Density Each Block Kg/m^3 (lbf/ft^3)	Dry Density Test Specimen Kg/m^3 (lbf/ft^3)	Elastic Limit Stress KPa (lbs/in^2)	Initial Tangent Young Modulus MPa (lbs/in^2)
EPS 40	I	16 (1.0)	15 (0.90)	40 (5.8)	4 (580)
EPS 50	VIII	20 (1.25)	18 (1.15)	50 (7.2)	5 (725)
EPS 70	II	24 (1.5)	22 (1.35)	70 (10.1)	7 (1015)
EPS 100	IX	32 (2.0)	29 (1.8)	100 (14.5)	10 (1450)

Table 9. Equivalent Soil Subgrade values of EPS-block geofoam for pavement design.

NCHRP 24-11 Designation	Minimum Allowable Full Block Density Kg/m^3 (lbf/ft^3)	CBR (%)	Young's Modulus MPa (lbs/in^2)	Resilient Modulus MPa (lbs/in^2)
EPS 50	20 (1.25)	2	5 (725)	5 (725)
EPS 70	24 (1.5)	3	7 (1015)	7 (1015)
EPS 100	32 (2.0)	4	10 (1450)	10 (1450)

Table 10. Expanded polystyrene (EPS) design and construction guidelines.

Design Parameters	
Density, dry	16-32 kg/m^3 (1-2 pcf).
Compressive and Flexural Strength,	Varies with density, 40-100 kPa (6-14 psi).
Modulus of Elasticity:	4-10 MPa (580-1450 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.(1)
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 75-100 kg/m^3 (4.8-6.4 pcf) after 10 years. Blocks above the water had densities of 30-50 kg/m^3 (1.9-3.2 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.(6)	
Use side slopes less than or equal to 2H:1V and a minimum cover thickness of 0.25 m (0.8 ft.). If a vertical face is needed, cover exposed face of blocks with shotcrete or other material to provide long-term UV protection.	

Table 10 (cont.)

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 6. 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.

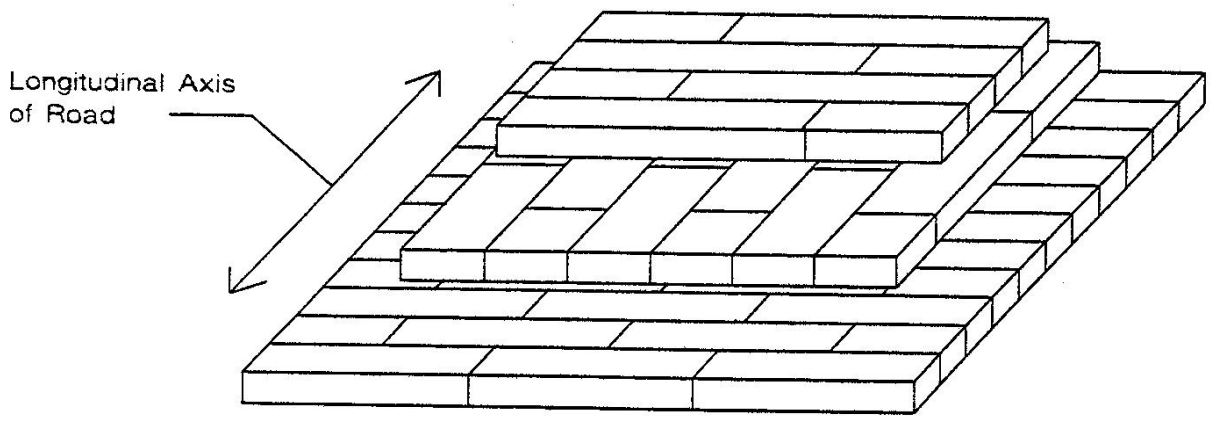


Figure 6. Isometric View of Typical EPS Block Layout.[6]

Foamed Concrete

Foam concrete is a liquid product that is practically self leveling, can be pumped over a distance as great as 1000 m (3300 ft.) ,and will begin to harden between 2 to 6 hours after production. Design with this product is analogous to design with conventional concrete, using the typical average properties summarized in Table 11 for a range of cast densities.[23] In computing buoyant stability, the low value of the volumetric mass should be used. For settlement calculations, the high value should be used with due consideration for water absorption. Some additional design considerations include the following[24]:

- 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.

Table 12 summarizes key design and construction considerations.

Table 11. Typical properties for foam concrete.[23]

	Foam Concrete Type (Density)							Unit
Density, cast	25	31	37	44	50	56	62	pcf
Density, cured, 28 days	23	29	36	42	48	55	61	pcf
Density, oven dry[1]	19	22	27	31	37	43	47	pcf
Average Compressive Strength, 28 days[2]	70	140	260	360	450	550	650	psi
Average Compressive Strength, 56 days[2]	90	170	300	400	580	650	750	psi
Tensile strength, 28 days, normal[3]	7	14	26	36	45	55	65	psi
Flexural strength, 28 days, normal[4]	12	25	47	65	81	99	117	psi
Young's modulus E, 28 days, normal[3]	$4.4x10^4$	$9.4x10^4$	$1.7x10^5$	$2.4x10^5$	$3.2x10^5$	$4.2x10^5$	$5.4x10^5$	psi
Freeze/thaw resistance at 100 cycles[5]	92	94	97	97	98			%
Freeze/thaw resistance at 300 cycles[6]	81	84	86	87	91			%
Shrinkage, 1 year, prism	0.65	0.55	0.45	0.45	0.4	0.4	0.35	%
Shrinkage, 1 year, practical	0.15	0.13	0.12	0.12	0.12	0.11	0.11	%
Hydraulic conductivity K p_c=2.5 psi[7]	$9x10^{-8}$	$4x10^{-8}$	$1x10^{-8}$	$8x10^{-9}$	$4x10^{-9}$			m/s
Hydraulic conductivity K p_c=18 psi[7]	$4x10^{-8}$	$1x10^{-8}$	$5x10^{-9}$	$3x10^{-9}$	$1x10^{-9}$			m/s
Water absorption[8]	75	50	33	22	15	10	7	k/m^2
Diffusion resistance μ								
+ R.H. = 50% to R.H. = 100%	2.5	3.5	4	4.5	5.5	6	6.5	--
+ R.H. = 70% to R.H. = 100%	5	6	7	8	9	10	12	--
Heat conductivity λ								
+ Oven dry	0.09	0.10	0.12	0.14	0.17	0.20	0.23	W/m.K
+ at R.H. = 70%	0.11	0.13	0.15	0.18	0.22	0.26	0.30	W/m.K
+ at R.H. = 95%	0.14	0.17	0.20	0.23	0.27	0.31	0.35	W/m.K

[1] Density when cured to constant mass at105° Celsius.

[2] Actual properties depend on mix designs, raw materials used, and manufacturing methods.

[3] Bending tensile strength and dynamic E-modulus determined with 2-point symmetrical bending test.

[4] With 4-point bending test.

[5] E_{dyn} as % of E_0 after 100 cycles according to ASTM C666, procedure B, modified for longer freeze/thaw cycle due to insulating properties of foam concrete.

[6] E_{dyn} as % of E_0 after 300 cycles according to ASTM C666, procedure B.

[7] K-value determined according to ASTM D 5084, method B, "falling head," using cell pressures or confining pressures of P_c = 2.5 psi & 18 psi.

[8] Total absorption in kg through 1 m^2 of foam concrete during 10 years exposure of that area to a constant head of 1 meter water.

[9] Heat conductivity (W/m.K) measurements are done with a Showa Denko K.K; the minimum value is valid for a moisture content of 70%, and the maximum value is valid for a R.H. of

Table 12. Foamed concrete design and construction guidelines.

Design Parameters	
Dry Density	320-970 kg/m^3 (20-61 pcf)
Compressive Strength:	Varies with density, 0.3-2.5 MPa (45-360 psi)
Water Absorption:	7-75 kg/m^2 (1.4-15 psf) depending on density
Freeze-thaw resistance, 100 cycles:	92-98 %, 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	
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 577 kg/m^3 (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 is 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.	
Lift thickness should be limited to 0.6 m (2 ft.) to allow workers to shape the surface while wading in the fluid mix. If shaping is not required, the lift thickness should not exceed 1.2 m (3.9 ft.), 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.	

CHAPTER 5
CONSTRUCTION SPECIFICATIONS AND CONTROL

For lightweight fill materials the typical specifications included in the Appendix are formatted as material requirements and construction methods.

5.1 SPECIFICATIONS

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

- *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, 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.

- *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 shredded tires.

For lightweight fill materials, such as EPS-block geofoam and foamed concrete, that have inherent strength, the compressive strength is dependent on the plant manufacturing process for geofoam, while that of foamed 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.

- the lift thickness for the foamed concrete, or the block dimensions for the geofoam.

Typical specifications for the following lightweight fills are given in Appendix A.

- Expanded Shale - Caltrans
- Geofoam Fill Material – Mass DPW (Central Artery)
- Foamed Concrete – Mass DPW (Central Artery)
- Wood Fiber - Washington DOT
- Shredded Tire - Model Specification
- Fly Ash - Delaware DOT

5.2 MONITORING AND CONSTRUCTION CONTROL

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, 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 shredded tires 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.

For projects where the shredded tires will be used, field monitoring should also be undertaken to confirm that there is not excessive steel wire with the shredded tires 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 has provided a classification for visually identifying different degrees of decomposition in the wood fiber.[17] Project personnel should be provided with this classification system so as to exclude the partially decomposed wood fibers from the new fill.

For the cohesive lightweight fill materials, field monitoring should include measurements of the density and compressive strength of the materials supplied. For geofoam, the density and compressive strength will be a function of the grade delivered with appropriate manufacturer QC documentation. Samples should be obtained for QA testing. Observations of the placements of the blocks should also be made to confirm that the blocks are placed without a continuous joint and that shear transfer plates are installed between successive lifts of the blocks. The geomembrane 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.

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.

The construction specifications provided in Appendix A detail the testing and construction control methods for each lightweight material.

CHAPTER 6
COST DATA

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 will increase the cost slightly.

- *Transportation costs.* If the project is located relatively far from the supply source, the transportation costs could be significant.

- *Availability of materials.* 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 or compaction costs* (could be higher than for soil fill). This would include moisture control for fly ash and boiler slag, geomembranes for geofoam, or greater numbers of 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. 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 cubic meter. The conversion from dollars per ton to dollars per cubic meter can be made with the following simplified expression:

$$\text{dollars per ton} \times \text{density, kg/m}^3 \times 1/910 = \text{dollars per m}^3$$

In this expression, the dollars per ton is frequently given as the price FOB at the plant. The transportation costs should then be added to this number.

Geofoam blocks are generally priced in dollars per board foot (1 ft by 1 ft by 1 in) so conversion to dollars per cubic meter is required ($0.15 per board foot = $63.50 per cubic meter).

Typical unit prices that are reported in the literature for various types of lightweight fill

materials are summarized in Table 13. In some cases, available cost information is limited, and the prices could vary significantly from that shown in the table. These prices were generated for projects constructed during 1993 to 1994, and could vary due to inflation, as well as other variables. The source of the cost is also included in Table 13. The geofoam and foamed concrete costs are more recent (1999-2002).

For a specific project, the engineer should contact local suppliers to obtain specific prices. Transportation costs could then be added, from which an estimate of the unit cost of the lightweight fill could then be obtained. Additional cost considerations would include compaction or specialty items, such as the need for geotextiles, geomembranes, drainage blankets, or soil cover. These costs may be significant.

Table 13. Typical costs for lightweight fill materials.

Lightweight Fill	Cost Per Cubic Meter	Source of Costs
Wood Fiber	\$12 to \$20[1] (\$9 - \$15/yd^3)	Reference 4
Air-cooled blast furnace slag	\$7.50 to \$9[2] (\$5.75 - \$7/yd^3)	S
Expanded blast furnace slag	\$15 to \$20[2] (\$11 - \$15/yd^3)	S
Boiler slag	\$3.00 to \$4.00[2] (\$2.30 - \$3.00/yd^3)	S
Fly ash	\$15 to \$21[2] (\$12.50 - \$16/yd^3)	S
Geofoam blocks (EPS)	\$40 to \$85[1] (\$30 - \$65/yd^3)	S, Reference 6
Foamed concrete	\$40 to \$55[3] (\$30 - \$42/yd^3)	S, Reference 7
Expanded Shale, Clay, and Slate (ESCS)	\$40 to \$55[2] (\$30 - \$42/yd^3)	S, Reference 7
Shredded tires	\$20 to \$30[1] (\$15 - \$23/yd^3)	Reference 20

[1] Price included transportation cost.

[2] FOB plant.

[3] Mixed at job site.

S Supplier of lightweight fill material.

CHAPTER 7
CASE HISTORIES

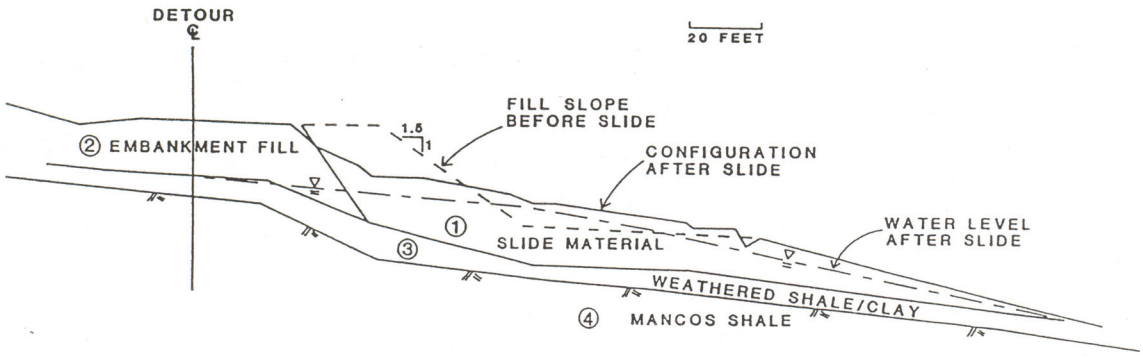
7.1 INTRODUCTION

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.

7.2 GEOFOAM FOR EMBANKMENT RECONSTRUCTION

In 1987, a portion of heavily traveled U.S. Highway 160 in southwestern Colorado experienced a slide failure that closed the eastbound lane over a length of about 61 m (200 ft.).[18] Figure 7 illustrates the conditions before and after the slide.



SOIL/ROCK PROPERTIES

SOIL/ROCK TYPE	TOTAL UNIT WEIGHT, PCF	MOISTURE %	L.L.	P.I.	C PSF	Ø
①	115	WET	–	–	50	15
②	115	16	35	14	500	20
③	120	14.1	36	15	1000	0
④	115	–	–	–	9000	0

Figure 7. Geometry and Cross-Section through Slide Area, Colorado Project.[41]

The subsurface conditions consisted of about 1.5-6 m (5-20 ft.) of a clayey highway embankment fill overlying 1-4 m (3-13 ft.) of natural clay. Both the embankment clay fill and the natural clay were moderately compact and of a stiff consistency. However, in the area of the slide, these deposits were very soft and saturated. Water levels measured during the field investigation were 0.6-0.9 m (2-3 ft.) below the surface at the slide area.

Below the overburden soils there is marine shale that is weak, easily eroded, and slippery when wet.

Slope stability analysis indicated that the slide was initiated by a rise of the water table in the embankment due to improper drainage. The roadway embankment blocked the previous flow path of water to the adjacent valley. This caused an increase in ground water level that had the effect of reducing the weight of the soil resisting the slide, and instability resulted.

Various alternatives that examined both cost and time were considered for restoration of the embankment. The first alternative involved removal of the entire slide debris and underlying weak soils and replacement with new fill. However, due to the presence of the old slides, right-of-way limitations, and the amount of earth involved, this option was deemed impractical and could not be constructed during the winter months.

The second alternative was to construct a rigid wall to retain the highway embankment. Several rigid wall systems supported on either drilled piers or driven piles were considered. This alternative was not selected, based on the high cost.

The slide correction method chosen, based both on cost and time, consisted of two portions. The time benefit was obtained from a winter construction schedule, which ensured project completion by the beginning of the summer peak travel season.

- An interceptor drain was installed on the uphill side of the roadway to collect ground water and discharge this water well beyond the slide area. The analysis indicated that if the water table could be lowered by 0.9 m (3 ft.), the safety factor would be increased by 20 percent.

- The factor of safety was further increased by the use of EPS-block geofoam in the upper portion of the slide zone as indicated in Figure 7. The material substitute reduced the driving forces. To prevent possible floating of the geofoam from ground water, a blanket drain was provided beneath the geofoam to conduct ground water away from this portion of the reconstructed area. A 250 mm (10 in.) perforated drainpipe was installed at the bottoms of the trench drains as shown in Figure 8.

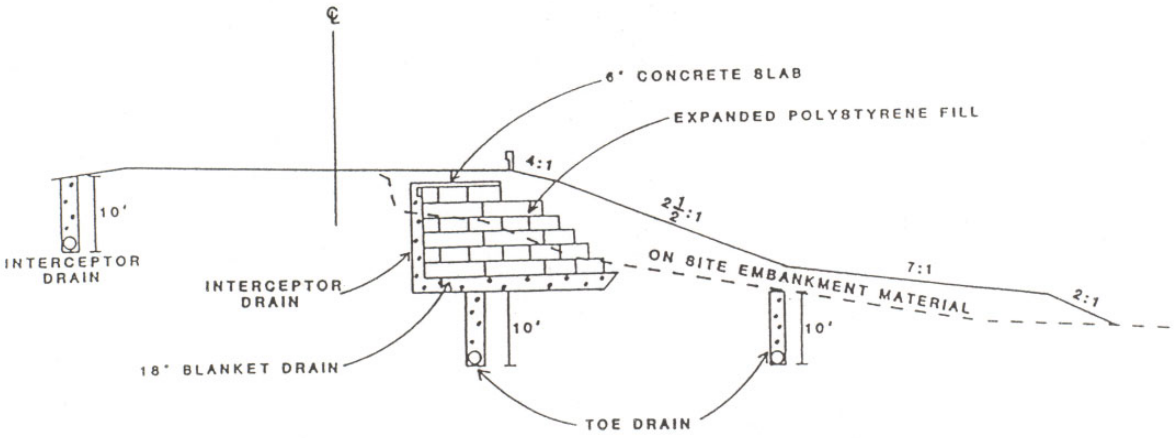


Figure 8. Designed Embankment and Drains - Colorado Project.

Figure 9. EPS Geofoam Blocks being Placed at Colorado Site.

To monitor the performance of the embankment, settlement plates were installed in locations directly over the geofoam. Settlement readings indicated there was 25-37 mm (1-1.5 in.) of compression of the geofoam as the soil overburden was placed on top of the geofoam during construction. Readings taken 1 year later indicated no additional movement, although this was a dry year. The second year after construction was much wetter, and an additional 13 cm of movement was observed. There was also some lateral movement detected, but no particular pattern could be determined. Figure 9 is a photo showing the geofoam blocks being placed.

7.3 WOOD FIBER EMBANKMENT ACROSS SOFT GROUND

Approximately 180 m (600 ft.) of two-lane roadway was constructed over swampy terrain in Washington State in 1987-88.[19] Figure 10 shows a generalized soil profile at the site, including the existing and final roadway grades.

The standard penetration resistance values in the organic sandy silt were generally less than 1. In place water contents ranged from 94-364 percent, with an average of 172 percent. The liquid limit ranged from 61-90 percent, and the plastic limit from 53-65 percent. The average unconsolidated, undrained strength determined from triaxial shear testing was 7.2 kPa.

The new embankment vertical grades required up to 13.4 m (44 ft.) 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 3.7 m (12 ft.). 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 13.4 m (44 ft.) of granular fill was expected to be 2.4 m (8 ft.), which was unacceptable because of the necessity of installing a culvert at the base of the fill.

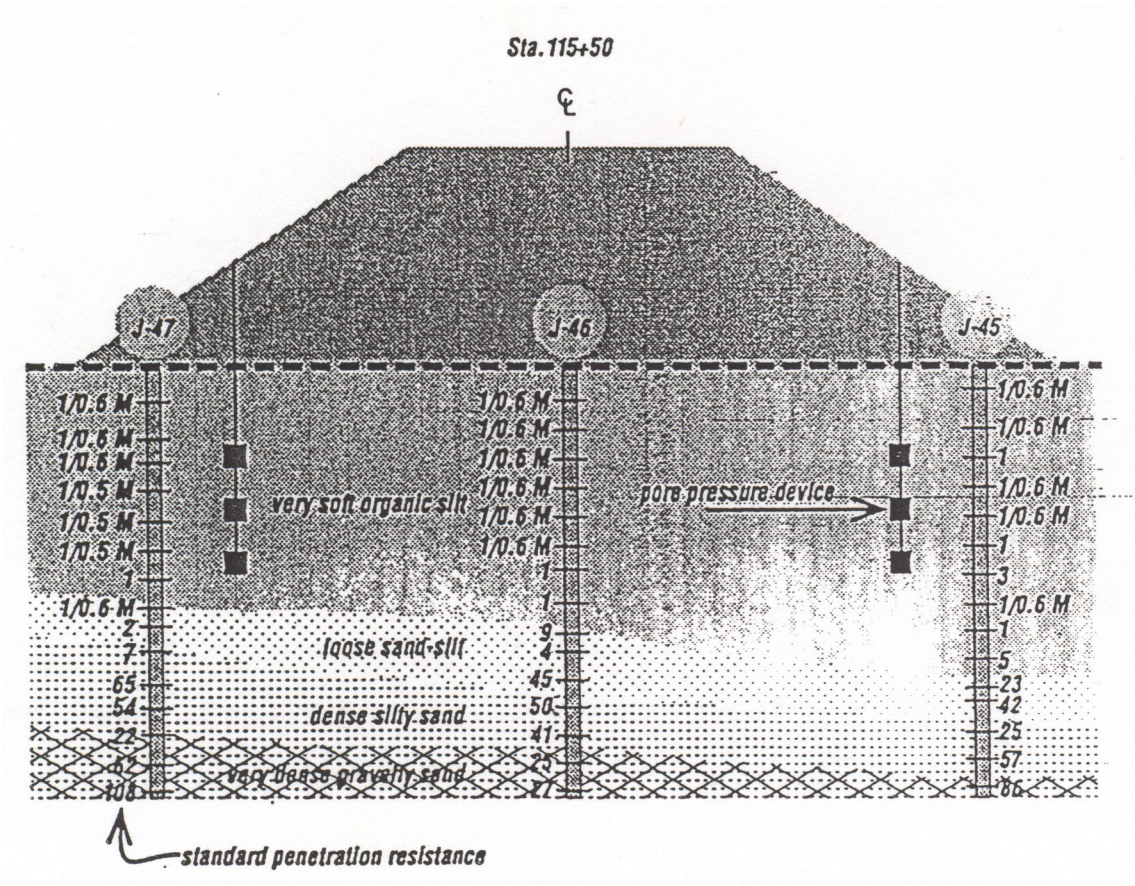


Figure 10. Soil Cross-Section at Wood Fiber Fill Location, Washington Site.[21]

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 1.5 m (5 ft.) of fill consisted of a silty, gravelly sand to raise the base of the embankment above the prevailing ground water table. The upper 1.2 m (4 ft.) of the fill was also a granular deposit to provide adequate subgrade support for the roadway. The remaining 10.7 m (35 ft.) of the embankment height consisted of the wood fiber-fill. For the stability calculations, the density of the wood fiber-fill was used as 640 kg/m^3 and an angle of 40°. The granular soils were assumed to have a unit weight of 2 Mg/m^3 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 [see Technical Summary #11 – Geosynthetic Reinforced Embankments over Soft Soils]. 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 1.5 m (5 ft.), plus an additional 0.13 m (0.4 ft.) 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 610 kg/m^3. Compaction was obtained by routing hauling equipment over the entire lift thickness of 0.3 m (1 ft.). The minimum mass of the hauling equipment was 15 Mg.

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 6.6 m and 9.5 m (22 ft. and 31 ft.). 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 1 m (3.3 ft.). Just prior to paving, this settlement had increased to 1.2 m (4 ft.). In September 1992, settlement increased to 1.4 m (4.6 ft.). This compares favorably with the estimated primary settlement of 1.5 m (5 ft.).

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.

7.4 WEIGHT COMPENSATION WITH FOAMED CONCRETE

The New York Department of Transportation used foamed concrete fill on 12 separate projects. The typical application was for balanced excavations.[7] 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, 1 m (3.3 ft.) of foamed concrete fill at a density of 480 kg/m^3 (3 pcf) will have the same weight as 0.25 m (0.8 ft.) of existing fill with a density of 1920 kg/m^3(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 1 m (3.3 ft.). This is not a large grade increase, but the embankments were underlain by 9-10 m (30-33 ft.) 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 2 m (6.6 ft.) below final grade was constructed of foamed concrete with a maximum unit weight of 480 kg/m^3. A drain was placed behind the existing abutment so as to reduce water pressures on the abutment. Figure 11 illustrates the final design.

Based on their experience with foamed concrete fills, the New York Department of Transportation developed the following guidelines:

- The foamed concrete should generally be placed in 0.3 m (1 ft.) maximum lifts. To establish a side or end slope, the fill can be placed in step lifts, then topped with conventional fill.

- To prevent water absorption or buoyancy, the foamed concrete should be placed above the normal ground water in high tide elevations. To compensate for occasional extremes in these elevations, the bottom placement area should be lined with a geomembrane. Placing underdrains at the base of curtain walls, wing walls, abutments, and at the pavement edge should enhance drainage.

- Freeze thaw was initially a concern, but if the foamed concrete is placed above the water table, very little water is likely to find its way into the fill. Furthermore, the normal sub-base in roadway pavements above the level of the fill acts as an insulation from freezing temperatures.

- In roadway areas, foamed concrete of a higher strength and higher density should be used in the top lift in order to better resist traffic loads.

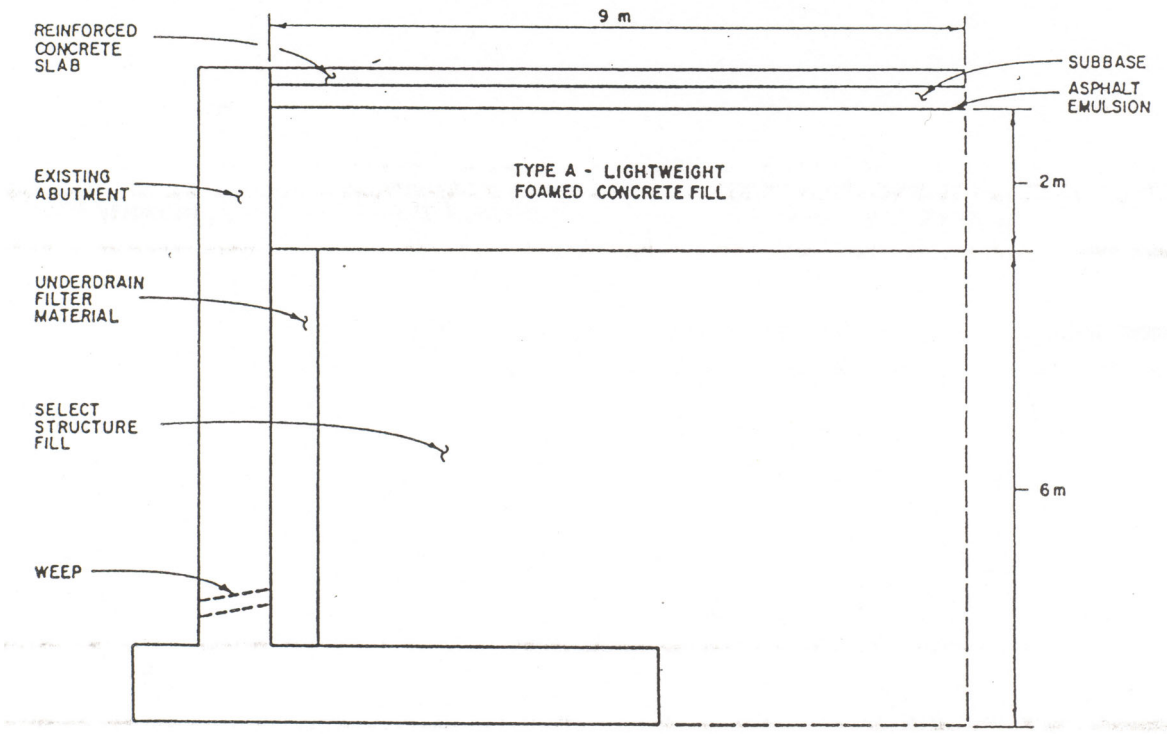


Figure 11. Foamed Concrete Fill behind Existing Abutment, New York Project.

- To prevent the pavement from shifting over the top of the fill if subjected to heavy traffic, an occasional key way is provided in the foamed concrete fill. The depth of the key way is on the order of 150 mm (6 in.) and is at least 1.5 m (5 ft.) wide.

- The cost in New York for these projects typically varied from \$50-\$70/m^3 (1981-1992).

7.5 Shredded Tires for Repair of a Landslide

An existing highway embankment in southern Oregon was widened 6.1 m (20 ft.) and raised 1.2 m (4 ft.).[20] The additional embankment load remobilized an old landslide that moved progressively down slope, perpendicular to the highway. The geotechnical investigation indicated that reducing the embankment load and adding a down-slope counterbalance could arrest the slide movement. The material for the counterbalance came from the soils that were excavated from the existing embankment. Various lightweight materials were considered for reconstructing the embankment, but shredded tires were selected because of the favorable material costs, including a State rebate for beneficial use. The typical cross section of the

repair scheme is shown in Figure 12. A rock drainage blanket was used beneath the shredded tire fill to isolate the tires from the ground water table. This was done for environmental protection purposes. The tire chips were positioned using dump trucks to place the fill at the end of a prepared area. A D8 dozer was then used to spread and compact the tire chips in 0.9 m (1 ft.) lifts. The dozer was routed back and forth, first in a longitudinal direction, then in a transverse direction; this comprised 1 coverage. At least 3 coverages were completed for each lift.

A geotextile was placed on top of the shredded-tire fill to separate the chips from the soil cap. A 0.9 m (1 ft.) soil cap was placed over the tire chip fill using 200 mm (8 in.) lift thickness. The soil was compacted to a minimum of 95 percent of standard proctor density, except for the first lift, which was 90 percent of density. During the first lift, the cap deflected significantly under compactive effort, but with each additional lift, the deflections became progressively smaller.

The major construction challenge was the impact of the wire strands exposed on the shredded tire chips. These frequently punctured the tires on the construction equipment, and prevented haul trucks from being routed over the fill. This problem might have been avoided if an inspector was at the chipping plant to ensure compliance with the maximum amount of steel shred exposure in the tire chips. Following construction of the pavements, it was found that one section had suffered severe rutting and cracking. The area was excavated to the top of the shredded-tire fill, and it was determined that the soil cap was only 0.5-0.6 m (1.6-2 ft.) thick. The shredded tires were then excavated to accommodate the full 0.9 m (3 ft.) soil cover, and the pavements were rebuilt.

Settlement plates were installed within the embankment with one plate at the bottom of the shredded-tire layer and one at the top. This allowed for calculation of the density of the shredded tires in several different conditions, including the density as compacted by the dozer, after surcharging by the soil cap and pavement section, and finally after 1 year of traffic. The densities calculated from these settlement surveys, plus measurements taken on the trucks as hauled to the site, are summarized in Table 14.

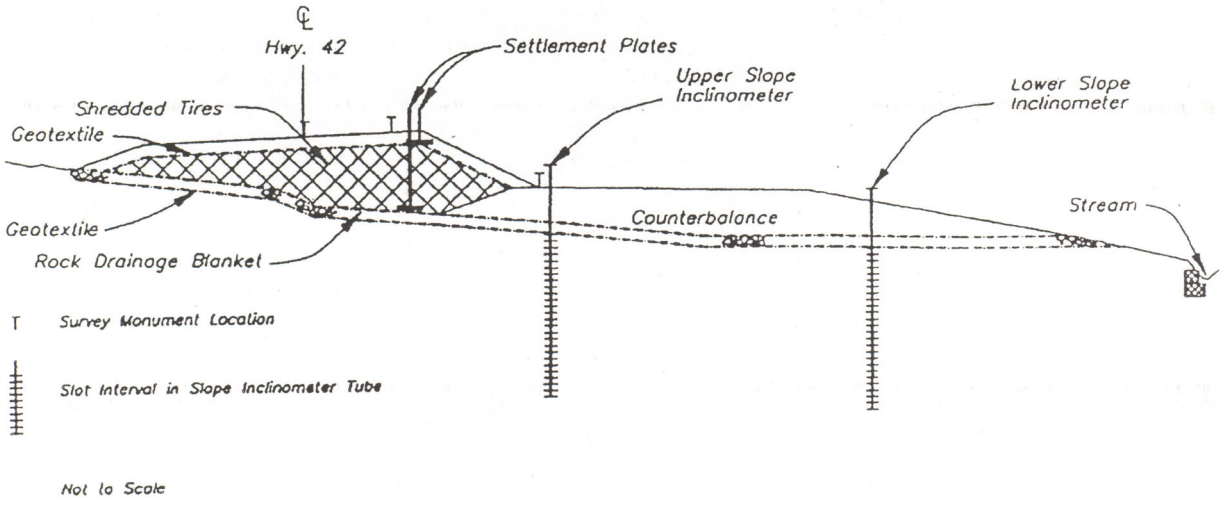


Table 14. Shredded tire densities.[20]

Condition	Density
Loose (as loaded in trucks)	390 – 485 kg/m^3
Loose (after 64 km haul in trucks)	535 kg/m^3
Compacted (after three dozer passes)	730 kg/m^3
Surcharged (after final pavement lift)	845 kg/m^3
Final (after 1 year of compression)	860 kg/m^3

Falling-weight deflectometer tests were performed on top of the aggregate base course after both the first and second lift of asphaltic concrete was placed. These tests showed that the shredded-tire fill appeared to represent a softer subgrade than the surrounding earth embankments. However, the deflection magnitudes measured on top of the asphaltic concrete met the Oregon Department of Transportation criteria for a 20-year pavement design-life.

The cost for delivering the shredded tires to this site was $33.00/Mg. However, the state rebated a cost of $22.00/Mg, which resulted in a net cost of $11.00/Mg. The cost of placing and compacting the shredded tires was $9.18/Mg. In terms of unit volume, the final cost, including compaction, came to $16.82/m^3 ($12.87/yd^3). If not for the reimbursement, the cost would have been $35.16/m^3 ($26.90/yd^3).

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APPENDIX A
TYPICAL SPECIFICATIONS

Selected material and construction specifications developed by various transportation agencies for various lightweight fill materials have been included in this Appendix. They often contain references to the specific agency standard construction requirements, which have not been included, and, therefore, would need to be modified by each user.

The following specifications are included in this Appendix:

- Expanded Shale – Caltrans (see www.escsi.org for additional spec recommendations)
- Geofoam Fill Material – Mass DPW (Central Artery) (Revised)
- Foamed Concrete – Mass DPW (Central Artery)
- Wood Fiber - Washington DOT
- Shredded Tire - Model Specification
- Fly Ash - Delaware DOT

Expanded Shale - Caltrans Specification

This work shall consist of furnishing, loading, hauling, placing, and compacting lightweight fill, as shown on the plans, and in conformance with these special provisions. Lightweight fill shall consist of a rotary kiln expanded shale aggregate of the extruded type. The shape of the extruded material shall be subangular to angular. Round shaped material shall not be allowed. When delivered to the project site, expanded shale aggregate shall conform to the following gradation:

Sieve Sizes	Percent Passing
19 mm	100
13 mm	90-100
9.5 mm	55-65
6 mm	5-15
#4	0-10
#30	0-5

The material shall have a maximum density of 673 kg/m^3 and a minimum density of 609 kg/m^3 determined in accordance with ASTM Designation C 29.

Lightweight fill shall be placed and compacted to designated dimensions as specified in Sections 19-1.03, "Grade Tolerance" and 19-6, "Embankment Construction" of the Standard Specifications.

The lightweight fill may be constructed by dumping from trucks or by any other method approved by the Engineer. Lightweight fill shall be placed in lifts of a maximum of one foot in depth of un-compacted material. Compaction shall be obtained by a minimum of one complete coverage of a D6 dozer or other tracked equipment having similar contact pressures.

The total quantity of lightweight fill will be computed in the same manner as specified for roadway excavation in Section 19-2.08, "Measurement" of the Standard Specifications, on the basis of the planned or authorized cross section for lightweight fill and the measured ground surface. No adjustment in the quantity of lightweight fill to be paid for will be made in the event that subsidence or consolidation occurs after the placing of lightweight fill material has begun.

The contract price paid per cubic meter for lightweight fill shall include full compensation for furnishing all labor, materials, tools, equipment, and incidentals, and for doing all the work involved in constructing the lightweight fill, complete in place, as shown on the plans, as specified in these special provisions, and as directed by the Engineer.

BLOCK-MOLDED EXPANDED POLYSTYRENE LIGHWEIGHT FILL (EPS-BLOCK FILL)

DESCRIPTION

1.01 GENERAL

A. This Section specifies the furnishing and installation of block-molded expanded polystyrene lightweight fill for use in bridge approaches and highway embankments referred to hereinafter as "EPS-Block Fill" structures on soft ground, as indicated on Contract Drawings. Provide all labor, materials and equipment necessary to complete the work of this Section.

1.02 RELATED WORK SPECIFIED ELSEWHERE

A. Carefully examine all of the Contract Documents for the requirements, which affect the work of this section.

B. Other sections of the Specifications, which directly relate to the work of this section include, but are not limited to, the following:

1. Section _____ - Excavation and Backfill
2. Section _____ - Ground Water Control
3. Section _____ - Lightweight Aggregate Fill
4. Section _____ - Grading
5. Section _____ - Reinforced Concrete Structures

1.03 DEFINITIONS

For the purposes of this specification, the following definitions are used for the parties indicated below, involved with EPS-Block Fill applications on this contract:

EPS-Block Fill Structure:
An assemblage of EPS blocks forming a single transition structure that constitutes part of this contract.

Molder:
The company actually manufacturing the EPS blocks used for the proposed EPS-Block Fill structures. In the event several EPS molders are supplying EPS blocks to this contract, one Molder shall be designated as the Primary Molder and assume the responsibility for all EPS block material supplied to this contract and its compliance with the requirements of this specification. All references hereinafter to the "Molder" shall be taken to mean the Primary Molder, if there are multiple molders.

Supplier:
The company having the contractual relationship with the Contractor for the supply of the EPS blocks. This may be the Molder (or Primary Molder in case of multiple molders) directly, or an intermediary company (typically a distributor of construction and/or geosynthetic products manufactured by others). Where appropriate, the Supplier may delegate certain tasks of this Specification to the Molder or Primary Molder.

EPS-block grade:
Refers to one of the NCHRP 24-11 material designation types indicated in Table 1 of this specification.

1.04 REFERENCES

A. GENERAL

Comply with the provisions of the following codes, specifications and standards except as otherwise indicated.

1. ASTM C203 - Breaking Load and Flexural Properties of Block-type Thermal Insulation.
2. ASTM C578 - Standard Specification for Rigid, Cellular Polystyrene Thermal Insulation.

1.05 SUBMITTALS

A. PRODUCT DATA

Molder's product data, installation instructions, use limitations and recommendations for each material specified herein shall all be submitted for review and approval by the Engineer. Certifications shall be provided stating that materials comply with requirements.

B. SHIPPING PLAN

As part of Phase I Manufacturing Quality Assurance (MQA) Supplier Pre-certification requirements under Section 1.06.B, the Contractor shall submit its proposed shipping procedure for the EPS block to the Engineer for review. Such procedure shall include protective measures during shipping to avoid any damage to the blocks, including crushing to the edges, sides, and corners of blocks. Timber cribbing with straps, tarp attached to the truck, or other effective means may be proposed to secure the blocks to the transporting vehicle. Alternatively, closed-container trucks may be used to transport the blocks to the Project construction site.

C. SHOP DRAWINGS

1. As described in Section 1.06.B, Contractor shall submit full-sized shop drawings to scale for each EPS-Block Fill structure on this contract, indicating the proposed location and layout of all EPS blocks, inter-block connectors, seismic connectors, and all accessory items to be used. The submitted drawings shall include, but not be limited to, plans, elevations, cross-sections showing profiles and cross-slopes, location of connectors between EPS blocks, connections, and accessory items, as necessary.

2. Contractor shall include on the submitted shop drawings a step-by-step description of the installation and construction procedure proposed for each EPS-Block Fill structure on this contract. Installation and construction sequence, supplemented by drawings as necessary, of the EPS blocks, pavement system, and utilities shall all be included. EPS block sizes and laying pattern, as well methods of temporarily ballasting and stabilizing EPS blocks to prevent movement during construction, prior to placement of the Load Distribution Slab, as applicable, shall all be identified.

3. Shop drawings, working drawings, installation and construction procedures, and supporting calculations shall all be stamped by a Professional Engineer registered in the Commonwealth of Massachusetts.

4. All details different than those depicted on the contract drawings, required in support of the construction procedures shall be engineered by the Contractor. All Contractor-engineered items shall be stamped by a professional engineer registered in the Commonwealth of Massachusetts. All such details shall be reviewed and approved by the Engineer prior to construction.

5. Review of the shop drawings and construction procedures shall be the primary responsibility of the Engineer, who may select to conduct parallel secondary reviews of shop drawings and construction procedures during the early phases of submittal reviews or for a selected number of EPS-block fill structures as appropriate. Contractor shall be notified of such reviews through standard Project procedures.

6. The layout of all EPS blocks and attachments shall be in conformance with the design details shown on the contract documents.

D. MANUFACTURING QUALITY CONTROL (MQC) SUBMITTALS

Submit the following in accordance with conditions of the Contract.

1. Test Compliance:

   a) The Contractor shall supply a summary of test compliance with specified performance characteristics and physical properties for the EPS blocks and galvanized steel connector plates for review and approval by the Engineer.

2. Certificates:

   a) The Contractor shall supply for review and approval by the Engineer, hard-copy product certificates showing compliance to the Material Properties stated in this specification. Product certificates shall be signed by the Supplier to certify material compliance with the specified performance characteristics, criteria, and physical requirements outlined in this specification.

   b) The Contractor shall indicate in writing to the Engineer whether or not the Supplier has a Third Party Certification. If Third Party Certification is offered, this notification shall be accompanied by documentation that indicates the business entity providing the Third-Party Certification and describes in detail the steps to be taken by this agency to verify the Molder's compliance with the specific requirements of this specification. Acceptance of the Molder's Third Party Certification by the Engineer will not waive the need for pre-construction product submittal and testing, as specified in Sections 1.06.B.2 and 1.06.B3.

   c) The Contractor shall submit, for the Department's acceptance, Supplier's standard warranty document or certificate executed by an authorized company official. Supplier's warranty is in addition to, and not a limitation of, other rights the Department may have under Contract Documents. Refer to Division I of these specifications for Project warranty provision.

1.06 MANUFACTURING QUALITY ASSURANCE (MQA)

A. GENERAL REQUIREMENTS

1. Manufacturing Quality Assurance (MQA) of the EPS-block product will be conducted to verify the Molder's Quality Control (MQC) procedures. The Engineer will have primary responsibility for all MQA, unless the Department notifies the Contractor otherwise. The Engineer will communicate directly only with the Contractor in matters and questions of MQA, unless all parties agree otherwise.

2. MQA of the EPS-block will consist of two phases:
   - Phase I MQA – Supplier Pre-certification, which consists of pre-certification of the Supplier and shall be conducted prior to shipment of any EPS blocks to the Project construction site. Phase I MQA is covered in Section 1.06.B of this specification.
   - Phase II MQA – Block Verification, which is conducted as the EPS blocks are delivered to the Project construction site. Phase II MQA is covered in Section 1.06.D of this specification.

3. Regulatory Requirements: Installation must comply with the requirements of all applicable local, state, and national jurisdictions.

4. Pre-installation Meeting: A pre-installation meeting shall be held to verify Project requirements, substrate conditions, and details relative to the manufacturing, shipping, and placement of the EPS-block. The meeting shall involve the Contractor, Engineer, and the EPS Supplier.

5. Construction Quality Control: Contractor shall be directly responsible for all

6. Construction Quality Control (CQC): Items covered by CQC include all earthwork and related activities, other than manufacturing and shipment of the EPS blocks.

7. Previously used EPS blocks are not allowed in part or in full on any and all EPS-Block Fill structures on this contract.

B. PHASE I MQA – SUPPLIER PRECERTIFICATION

No EPS blocks shall be shipped to the Project construction site until such time as all parts of Phase I MQA – Supplier Pre-certification, as specified in this section, have been completed in the order listed below:

1. The Contractor shall supply a scale with sufficient capacity and precision for weighing of the EPS blocks. This scale shall be delivered to the Project

construction site or to an alternate location specified by the Engineer. This scale shall be recently calibrated and certification of such calibration made available to the Engineer.

2. The Contractor shall deliver a minimum of three full-sized EPS blocks for each NCHRP 24-11 EPS-block grade to be used on this contract to a location specified by the Engineer. These blocks shall in all respects be the same as the blocks to be supplied to this contract, including required seasoning as described in Section 1.07.C.

3. The Engineer will weigh and measure each of the three blocks of each grade of EPS supplied. The Engineer will sample and test at least one of the three blocks of each grade, selected randomly, to evaluate the ability of the supplier to deliver EPS blocks of quality as specified herein. The sampling and testing protocol will be the same as for Phase II MQA as discussed in Section 1.06.D. Any EPS blocks not used for testing may be utilized for construction provided they satisfy all the requirements outlined in this specification.

4. The Contractor has submitted shop drawings as required by Section 1.05.C of this specification, and such drawings have been reviewed and approved for construction in conformance with Project standards.

5. Prior to delivery of any EPS blocks to the Project construction site, a meeting shall be held, as a minimum, between the Engineer and Contractor. The Supplier and/or Molder of the EPS-blocks may also attend at the Contractor's discretion to facilitate answering any questions first hand. The purpose of this meeting shall be to review the Phase I MQA results and discuss Phase II MQA as well as other aspects of construction to ensure that all parties are familiar with the requirements of this specification. At the satisfactory conclusion of this meeting, the Contractor shall be allowed to begin on-site receipt, storage (if desired) of the EPS-blocks in accordance with Section 1.06.D of this specification.

C. PRODUCT DELIVERY, STORAGE, AND HANDLING

Prior to delivery of the EPS block to the Project construction site, Contractor shall review, plan, and implement, with the assistance of the Supplier, a material handling procedure that shall include the following as a minimum:

1. Care should be exercised during shipping to prevent any damage to the EPS blocks. Particular attention should be paid to avoid crushing to the edges, sides, and corners of the blocks during shipping from the Molder to the Project construction site. All shipments shall conform to the shipping plan, as indicated in Article 1.05.B.

2. The Contractor shall prevent any damage to the EPS blocks during delivery, handling, storage, and construction. EPS blocks with cracks of any size are considered not acceptable and shall be rejected and replaced by the Contractor with undamaged equal EPS blocks at no additional cost to the Department. Holes shall not be created in the blocks at any stage of manufacturing or construction to facilitate shipping or handling of the blocks during placement.

3. Each EPS block shall be labeled to indicate the name of the Molder (if there is more than one for a given EPS-Block Fill structure), the date the block was molded, the mass of the entire block in pounds, as measured after a satisfactory period of seasoning as specified in Section 1.07.D, the dimensions of the block in inches, and the actual dry unit weight in pounds per cubic foot.

   Additional identification markings using alphanumeric characters and/or symbols, applied as necessary by the Supplier, to indicate the location of placement of each block relative to the shop drawings shall also be provided. Stripes of different paint color can be utilized to identify blocks of higher grade EPS, if multiple grades of EPS are to be supplied for a given EPS-Block fill structure. If two grades of EPS blocks are to be supplied, a lack of marking shall be considered an acceptable marking for one of the material grades, as long as it is used for the lower (lowest) grade EPS blocks supplied for that structure. Any paint, etc., used to mark EPS blocks shall be chemically compatible with EPS, and shall not cause any dissolution of the EPS during application of the mark.

4. If the EPS blocks are to be stockpiled at the construction site until placement, a secure storage area shall be identified and designated by the Contractor for this purpose. The storage area shall be away from any heat source or construction activity that produces heat or flame or would expose the blocks to hydrocarbon fuels (diesel, kerosene, gasoline). In addition, personal tobacco smoking shall not be allowed in the storage area. EPS blocks in temporary on-site storage shall be secured with sandbags and similar "soft" weights to prevent their being dislodged by wind. The blocks shall not be covered in any manner that might allow the build up of heat beneath the cover. The blocks shall not be trafficked by any vehicle or equipment. In addition, foot traffic by persons shall be kept to a minimum.

5. EPS is not an inherently dangerous or toxic material, so there are no particular safety issues to be observed other than normal construction safety and protection against heat and flame and hydrocarbon fuels (diesel, kerosene, gasoline), which can cause EPS to melt. However, extra caution is required during wet or cold weather since surfaces of the EPS blocks tend to be more slippery when wet than dry. In addition, when air temperatures approach or go below freezing, a thin layer of hoarfrost (ice) can readily develop on the exposed surfaces of EPS blocks if the dew-point is sufficiently high. Thus, the surfaces of the EPS blocks can pose particular slip hazards in this condition. The maximum amount of time that

EPS blocks can be stored at the Project construction site shall be limited to 30 calendar days.

D. PHASE II MQA – BLOCK VERIFICATION

The Engineer shall assume the primary responsibility for conducting this phase of the work. Contractor shall cooperate with and assist the Engineer in implementing Phase II MQA – Block Verification.

No EPS blocks shall be placed on any structure of this contract until such time as all activities of Phase II MQA – Block Verification, as specified in this section, have been completed successfully in the order listed below:

1. Each block of every grade of EPS delivered on any truck to the construction site shall be inspected on-site visually to check for damage, as well as for verification of the labeled information included on each block. Any blocks with damage or not meeting the requirements of the specifications will be rejected on the spot.

2. At least one block of every grade EPS delivered on every truck to the construction site shall be checked to verify its compliance with the requirements of the specifications for the minimum block dry unit weight specified in 2.01.A, as well as the physical tolerances, specified in sections 2.01.D through 2.01.F, inclusive. Weighing of blocks shall be conducted onsite using a scale with sufficient capacity and precision for weighing of EPS blocks to be supplied by the Contractor in conformance with Section 1.06.B.1.

3. Should verification of the parameters of item 2 above indicate lack of compliance, at least three additional blocks of every grade EPS delivered from the same truck-load will be individually checked. The entire shipment of the grade of EPS in question shall be rejected should any one of the three additional blocks fail to meet the requirements of the specifications for minimum block dry unit weight and physical tolerances outlined above.

4. At least one block of every grade of EPS delivered on the first truck to the construction site for use on any one EPS-Block Fill structure, shall be selected for sampling and testing. Sampling will be at locations A, B and C shown in Figure 1. The samples should be approximately square in cross-section and of sufficient width to enable preparation of the test specimen required by this specification. The

5. Contractor shall cooperate with and assist the Engineer with obtaining the necessary samples. Testing will be performed by or under the direction of the Engineer.

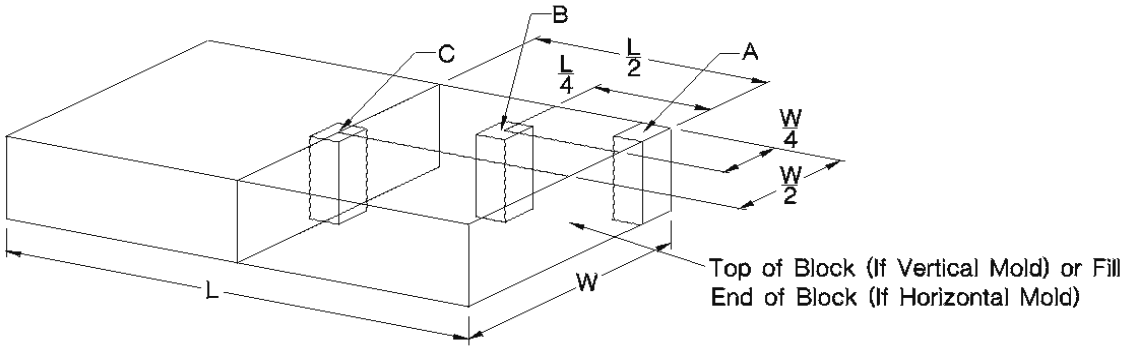

Figure 1. Locations of Required EPS Block Sampling.

Laboratory tests will check for compliance with the Material Properties shown on Table 2 of this specification. Additional blocks of each grade of EPS shall be selected by the Engineer for sampling during the course of construction at a rate of sampling not to exceed one block for every 500 cubic yards of EPS delivered.

6. Portions of sampled blocks that are used for testing are not acceptable for construction. Portions of sampled blocks that are otherwise acceptable can be used as desired by the Contractor, provided that they comply with all other requirements of this specification.

7. The Contractor shall allow for early delivery of the EPS blocks to the construction site to allow for laboratory testing of the blocks. For those truckloads where EPS blocks will be selected for sampling and testing per section 1.06.D.4, a minimum of 3 business days are required prior to their scheduled installation, to allow for samples to be taken and laboratory testing conducted. Any shipment of EPS blocks, for which the representative samples fail to meet the parameters outlined in this specification, are considered defective and shall be replaced by the Contractor with non-defective EPS block at no additional cost or time to the Department.

1.07 MANUFACTURING QUALITY CONTROL (MQC)

A. Manufacturing Quality Control (MQC) of EPS block product is the primary responsibility of the Molder. The purpose of this section is to define the parameters for use by a Molder in developing an MQC plan. These parameters will also be those measured as part of the Manufacturing Quality Assurance (MQA) to be conducted by the Engineer. MQA requirements are detailed in Section 1.06 of this specification.

B. All EPS blocks shall consist entirely of expanded polystyrene. All EPS block shall consist of virgin raw material (expandable polystyrene bead or resin). Previously

used EPS blocks are not allowed in part or in full on any and all EPS-Block Fill structures on this contract.

C. All EPS blocks shall be adequately seasoned prior to shipment to the Project construction site. For the purposes of this specification, seasoning is defined as storage in an area suitable for the intended purpose, as subsequently defined herein, for a minimum of 72 hours after an EPS block is released from the mold. Seasoning shall be done within a building or other structure that protects the EPS blocks from moisture as well as UV radiation. The area in which EPS blocks are stored for seasoning shall also be such that adequate space is allowed between blocks and positive air circulation and venting of the structure be provided so as to foster the out-gassing of blowing agent and trapped condensate from within the blocks. The Engineer shall be allowed to inspect the structure to be used for seasoning upon request and during normal business days and hours. The supplier may request a shortened seasoning period if the EPS blocks are seasoned within an appropriate heated storage space, and the Molder demonstrates to the satisfaction of the Engineer that the alternative seasoning treatment produces blocks that equal or exceed the quality of blocks subjected to the normal 72-hour seasoning period.

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D. All EPS blocks shall satisfy the product flammability requirements specified in ASTM C 578.

E. All EPS blocks shall satisfy the Material Property requirements outlined in Section 2.01 of this specification.

F. All galvanized steel inter-block connectors shall satisfy the requirements outlined in Section 2.02 of this specification.

1.07 PROJECT CONDITIONS

A. Provide inserts and anchorages that must be built into other work at the time they are needed.

MATERIALS

2.01 EXPANDED POLYSTYRENE BLOCK

A. Only NCHRP 24-11 material type designations shall be used on all correspondence and communication related to this contract. Table 1 indicates the NCHRP 24-11 material type designations used for the different unit weights of EPS blocks that are covered by this specification. For a given material type, the dry unit weight of each EPS block (as measured for the overall block as a whole) after the period of seasoning, as defined in Section 1.07.C of this specification, shall equal or exceed that shown in Table 1. The dry unit weight shall be determined by measuring the

mass of the entire block by weighing on a scale and dividing the mass by the volume of the block.

B.

Table 1. NCHRP 24-11 material type designations for EPS-block.

NCHRP 24-11 Material Designation	Minimum Allowable Dry Unit Weight of Entire EPS Block (lbs/ft^3)
EPS40	1.00
EPS50	1.25
EPS70	1.50
EPS100	2.00

C. Table 2 gives the minimum allowable values for the Material Properties corresponding to each NCHRP 24-11 material type shown in Table 1. These material properties shall be obtained by testing specimens prepared from samples taken from actual blocks produced for the Project covered by this specification for either MQC by the Molder or MQA by the Engineer as described in Section s 1.07.D and 1.06.D, respectively, of this specification. Testing for material properties shall comply with the following:

- All test specimens shall be seasoned as specified in ASTM C 578.
- Dry density, compressive strength, and flexural strength shall all be measured, as specified in ASTM C 578.
- The specimens used for compressive testing shall be cubic in shape with a two (2) inch side length.
- A strain rate of 10% per minute shall be used for the compressive strength tests.
- Both the elastic-limit stress and initial tangent Young's modulus shall be determined in the same test used to measure compressive strength. The elastic-limit stress is defined as the measured compressive normal stress at a compressive normal strain of 1%. The initial tangent Young's modulus is defined as the average slope of the compressive stress versus compressive strain curve between 0% and 1% strain.

Table 2. Minimum allowable values of MQC/MQA material properties for individual test specimens.

NCHRP 24-11 Material Designation	Dry Density (lbs/ft^3)	Compressive Strength (psi)	Flexural Strength (psi)	Elastic Limit Stress (psi)	Initial Tangent Young's Modulus (psi)
EPS40	0.90	10	25	5.8	580
EPS50	1.15	13	30	7.2	725
EPS70	1.35	15	40	10.2	1015
EPS100	1.80	25	50	14.5	1450

D. Each EPS block shall meet dimensional tolerances, as determined in three distinct areas:
   - Variations in linear dimensions as defined in Section 2.01.D.
   - Deviation from perpendicularity of block faces as defined in Section 2.01.E.
   - Overall warp of block faces as defined in Section 2.01.F.

E. The thickness, width, and length dimensions of an EPS block are defined herein as the minimum, intermediate, and maximum overall dimensions of the block, respectively, as measured along a block face. These dimensions of each block shall not deviate from the theoretical dimensions by more than ± 0.5%.

F. The corner or edge formed by any two faces of an EPS block shall be perpendicular, *i.e.*, form an angle of 90°. The deviation of any face of the block from a theoretical perpendicular plane shall not exceed one eighth (1/8) inch over a distance of twenty (20) inches.

G. Any one face of a block shall not deviate from planarity by more than one quarter (1/4) inch when measured using a straightedge with a length of ten (10) feet.

H. The EPS shall contain a flame retardant additive and shall have UL Certification of Classification as to External Fire Exposure and Surface Burning Characteristics so as to comply with the Oxygen Index requirements of ASTM C 578.

2.02 INTER-BLOCK CONNECTORS

A. Inter-block Connectors shall be used to restrain EPS blocks from moving laterally in layer over layer applications. Contractor shall supply samples and test data for the proposed inter-block connectors to comply with Section 2.02.B of this specification for review and approval by the Engineer prior to construction.

B. Inter-block Connectors shall be made of 20 gauge (minimum) galvanized steel with two-sided multi-barbed design, or approved equal, capable of piercing the EPS up to three-quarter (3/4) inch. Each plate shall be capable of a lateral holding strength of at least 60 lbs. Refer to Contract Drawings for suggested layout and number of Inter-block Connectors required.

C. Should the size of supplied EPS block be different than that shown on the Contract Drawings, Contractor shall propose a suggested layout for the inter-block connectors for review and approval by the Engineer prior to the start of construction. The revised payout shall be incorporated on the shop drawings in conformance with Section 1.05.C of this specification.

3.01 CONSTRUCTION QUALITY CONTROL (CQC)

A. The Contractor shall be directly responsible for all Construction Quality Control (CQC). Items covered by CQC include all earthwork and related activities other than manufacturing and shipment of the EPS blocks. Items of particular relevance to the placement of EPS block fill structure are given in sections 3.02 through 3.05, inclusive, of this specification.

B. The Engineer will be responsible for providing Construction Quality Assurance (CQA) of the Contractor's construction activities.

3.02 SITE PREPARATION

A. Where required by the Contract Drawings, the natural soil subgrade shall be excavated to the elevations given on the Contract Drawings.

B. The natural soil subgrade shall be cleared of vegetation, any large or sharp-edged soil particles, any kind of debris, and be reasonably planar prior to placing a geotextile and sand bedding layer. Reasonably planar is defined as one (1) inch over ten (10) feet.

C. The sand bedding layer shall consist of sand borrow conforming to M1.04.0, Type b in accordance with Section 170. The required smoothness of the sand bedding layer prior to placement of the first layer of EPS blocks shall be no more than ± three-eighth (3/8) of one (1) inch over any ten (10) ft. distance.

D. There shall be no debris of any kind on the sand bedding surface at the time EPS blocks are placed.

E. Unless directed otherwise by the Engineer, there shall be no standing water or accumulated snow or ice on the sand bedding layer within the area where EPS blocks are placed at the time of block placement.

F. EPS blocks shall not be placed on a frozen subgrade nor de-icing slats be used except as directed by the Engineer.

3.03 EPS BLOCK PLACEMENT

A. EPS blocks shall be placed at the locations shown on approved shop drawings submitted by the Contractor. Particular care is required if EPS blocks of different density are to be used on EPS-Block Fill structures of this contract.

B. There shall be no debris of any kind between adjacent surfaces of EPS blocks at the time adjacent EPS blocks are placed.

C. There shall be no standing water or accumulated snow or ice on the previously placed EPS block layer within the area where subsequent EPS blocks are to be placed at the time of block placement.

D. EPS blocks shall be placed so that all vertical and horizontal joints between blocks are tight.

E. While placing successive layers of EPS blocks, Contractor should exercise care to guarantee that all placed blocks are supported over their entire bearing area. In the event the top constructed surface of an assembly of blocks becomes uneven or where rocking of the blocks is observed, Contractor shall notify the Engineer and propose a remedial procedure for corrective action. Such procedure shall be submitted for review and approval by the Engineer prior to resuming construction.

F. Blocks shall be placed such that the resulting exterior surfaces on the sides of the EPS-Block Fill structures are vertical and planar within a tolerance of one-eighth (1/8) inch between blocks. Block faces not satisfying this criterion shall be field trimmed using a hot wire cutting apparatus to achieve the desired evenness within the above tolerance.

G. The inter-block connectors shall be placed at the locations shown on the shop drawings and shall be set into the EPS block such that the inter-block connectors do not cause a gap to exist between adjacent layers of EPS blocks.

H. The final surface of the EPS blocks shall be covered as shown on the Contract Drawings. Care shall be exercised during placement of the cover material so as not to cause any damage to the EPS blocks.

3.04 FIELD QUALITY REQUIREMENTS

A. Field cutting of EPS blocks shall be permitted, but limited to the use of the following devices:

1. Hot wire cutters
2. Wire saws
3. Chain saws

B. With the exception of sand bags or similar "soft" weights used to temporarily restrain EPS blocks against wind, no construction material other than that shown on the contract drawings shall be placed or stockpiled on the EPS blocks.

C. At no time shall heat, open flame, or motor vehicle fuels be used in proximity to the EPS blocks so as to cause combustion or melting of the EPS.

D. The final surface of the EPS blocks shall be covered with a cast-in-place concrete

Load Distribution Slab as shown on the Contract Drawings. Care shall be exercised during placement of the cover material so as not to cause any damage to the EPS blocks. The surfaces of the EPS blocks shall not be directly traversed by any vehicle or construction equipment during or after placement of the blocks.

3.05 PAVEMENT CONSTRUCTION

A. The pavement system is defined for the purposes of this specification as all material placed above the EPS blocks within the limits of the roadway, including any shoulders.
B. The pavement system shall be constructed above the EPS blocks as shown on the contract drawings. Specifications covering construction of the pavement system are given elsewhere in the contract documents.
C. Concrete and reinforcement steel for the Load Distribution Slab shall be installed over the EPS blocks or separation layer using appropriate labor and equipment that will not damage the EPS blocks.

3.06 PROTECTION

A. Protect partial installation and installed product and finish surfaces from damage during construction in accordance with Section 1.06.C.

3.07 EPS DISPOSAL

A. Contractor shall assume total responsibility for disposal of EPS block material or portions of unused blocks resulting from testing or construction by returning it to the Supplier/Molder for recycling. Such process shall be conducted on a regular basis or as instructed by the Engineer.

COMPENSATION

4.01 METHOD OF MEASUREMENT

A. Supply and installation of EPS will not be measured separately.

4.02 BASIS OF PAYMENT

A. Payment for the work of this Section will be included as part of the Contract Price for the structure for whom it forms a part.

Foamed Concrete – Mass DPW (Central Artery)

LIGHTWEIGHT FOAM CONCRETE FILL
DESCRIPTION

1.01 GENERAL

A. This Section specifies requirements for Lightweight Foam Concrete Fill (LFCF).

B. Related Work specified elsewhere:
(Fill in as appropriate)
Section _____ - Excavation and Backfill
Section _____ - Groundwater Control
Section _____ - Support of Excavation
Section _____ - Embankment
Section _____ - Grading
Section _____ - General Requirements for Utility Work
Section _____- Precast Concrete Manholes, Structures, and Catch Basins Section Subdrains
Section _____ - Membrane Waterproofing

1.02 QUALITY ASSURANCE

A. The LFCF installer shall be certified by the manufacturer of the foaming agent and regularly engaged in the production and placement of LFCF. This shall include the completion of lightweight foam concrete fills having a minimum of 10,000 total cubic yards in the past two years. Furthermore, the material shall have been successfully applied on at least three (3) Engineered Fill projects that have performed satisfactorily for at least three (3) years.

B. For this portion of the work, the certified installer shall be pre-qualified and approved in writing by the foam agent manufacturer of the LFCF material.

C. The specialized batching, mixing, and placing equipment shall be automated and certified for the purpose by the manufacturer of the LFCF material. Bulk cement shall be weighed on a scale, which operates within a tolerance of one and one-half percent (1-½ percent) per batch. Cement and water may be premixed and delivered to the site. Foam shall be added and mixed at the site using the aforementioned equipment.

D. Use adequate numbers of skilled workers, who are thoroughly trained and experienced in the necessary crafts and who are familiar with the specified requirements and the methods needed for proper performance of the work of this Section.

E. The manufacturer's representative shall be experienced in placement of LFCF and shall be on site full-time during placement.

1.03 SUBMITTALS

A. The Contractor shall submit to the Engineer, the following product data:
   1. Material list of items proposed to be provided under this Section.
   2. Manufacturer's specifications, catalog cuts, and other engineering data needed to demonstrate compliance with the specified requirements.
   3. Mix designs for the LFCF, prepared by the manufacturer, showing compliance with the specified properties.
   4. Written evidence of acceptance of the certified producer/supplier by the manufacturer.
   5. Manufacturer's literature that indicates that the waterproofing material is compatible with lightweight foam concrete fill and will withstand the placement of the pavement section.
   6. Sequencing plan indicating the construction sequence for constructing the entire fill section, including piles, retaining walls, utilities, LFCF, and roadway surface.

B. At least two weeks prior to placing, submit twenty (20) 3-inch diameter by 6-inch high cylinder samples of a designed LFCF. The accepted samples shall become that standard of the material furnished under this Contract.

MATERIALS

2.01 MATERIALS

A. Comply with the following material requirements:
   1. A foaming agent from the selected manufacturer that will produce an LFCF material that complies with the specifications listed in Section 2.01.B.
   2. Type II Portland Cement shall be used and shall comply with ASTM C150.
   3. Admixtures for water reducing, retarding, accelerating, improving the bond, and for other specific properties may be used when specifically designed by the manufacturer of the LFCF selected.
   4. Water shall be potable and free from deleterious amounts of alkali, acid, and organic materials, which would adversely affect the setting time or strength of the LFCF.
   5. Subdrains shall be in accordance with Section *(fill in as appropriate*).

B. LFCF- Comply with the following requirements:

	Class A	**Class B**
Cast Density	28 – 32 pcf	36 – 40 pcf
Minimum - Dry Density	25 pcf	30 pcf
Minimum Unconfined Compressive Strength	40 psi	100 psi
Freeze Thaw Resistance (Min. Relative Young's Modulus, E = 80%)	100 cycles	300 cycles

C. Waterproofing System
   1. Elastomeric membrane: See specification Section *(fill in as appropriate)*
   2. Sheet membrane: See Specification Section *(fill in as appropriate)*
   3. Drainage composite: Prefabricated drainage composite conforming to the following minimum requirements:

Drain Core:

Thickness:	0.38”
Compressive Strength:	15,000 psf
Flow:	15 gpm/ft @ 3600 psf

Filter Fabric:

Equivalent Opening Size:	70
Flow:	160 gpm/sq. ft.
Weight:	3.1 oz.

CONSTRUCTION METHODS

3.01 SUBGRADE CONDITIONS

A. Examine the areas and conditions under which work of this Section will be performed. Correct conditions that will be detrimental to timely and proper completion of the work. Do not proceed until unsatisfactory conditions are corrected. Subgrade shall be graded and compacted to the lines and grades shown on the Drawings.

3.02 PRODUCT HANDLING

A. Protect the materials of this Section before, during, and after installation, and protect the work and materials of all other trades.

B. In the event of damage, immediately make replacements and repairs to the acceptance of the Engineer and at no additional cost to the Department.

3.03 INSTALLATION

A. The areas to be filled shall not have any standing water in them prior to placement of the LFCF. The Contractor shall ensure that the LFCF remains above the water table at all times during construction.

B. Any items to be fully or partially encased in the LFCF shall be properly set and stable prior to the installation of the LFCF.

C. Care should be taken to avoid freezing before initial set of the LFCF. If the LFCF must be placed in freezing conditions, the manufacturer shall be consulted as to what precautions are necessary to assure installation of an acceptable LFCF.

   LFCF shall not be placed at a temperature of less than 32 degree Fahrenheit, nor when freezing conditions are expected in less than 24 hours, unless precautions are taken to maintain temperatures above freezing. Do not place on frozen ground.

D. LFCF shall only be proportioned, mixed, and placed using equipment approved by the manufacturer. Once mixed, the concrete shall be conveyed promptly to the location of placement without excessive handling.

E. LFCF shall be placed in lifts not exceeding 2 feet in depth.

F. Scarify each lift before placing the next lift. Each lift shall be scarified to a minimum depth of ½ inch using a hand rake or other suitable means. Scarifying shall be done after sufficient curing time such that foot traffic will not excessively damage the lift surface (no greater than ¼-inch indentation).

G. Allow a minimum of two days between subsequent lifts. Prior to verification of the minimum specified compressive strength by the Engineer's testing program, additional lifts may be placed after the two-day minimum at the Contractor's risk. Any material that does not meet the minimum specified strength within 28 days shall be removed and replaced by the Contractor at no additional cost.

H. Move the discharge hose(s) sufficiently to ensure level filling through the specified fill areas. Uneven filling is not permitted.

I. Limit the area of pouring to the volume that can be placed within one hour, up to the maximum two-foot lift height. Stagger pours such that vertical joints are at least two feet apart.

J. Hose length shall not exceed 800 feet.

K. The final surface finish of LFCF shall be within ±0.1 feet of the elevations shown on the plans. The top two pours (4 feet) at the interface with the LFCF shall be cast vertical and plum to the dimensions shown on the plans.

L. Paving machines or other unusual loading of the LFCF shall not be permitted until the LFCF has attained the minimum specified compressive strength.

M. Any material that does not meet the criteria specified herein (including, but not limited to, Article 2.01.B) shall be removed and replaced at no additional cost.

N. The waterproofing system shall be installed as shown on the Drawings and in accordance with the manufacturer's recommendations. Two coats of waterproofing shall be applied.

O. The surface of the LFCF shall be stepped as shown on the plans, with a step height 2-½ inches (minimum) to ½-inch (maximum). Run of step shall be sloped at a minimum of 1 percent to provide positive drainage.

P. The prefabricated drainage composite shall be installed with the filter fabric facing the LFCF. The drainage composite shall be attached to the concrete retaining wall with adhesive as specified by the manufacturer. Nailing of the drainage composite will not be permitted.

3.04 OBSERVATION AND TESTING

A. All work performed under this Contract shall be subject to observation and tests by the Engineer. The Contractor shall cooperate with the Engineer and shall furnish all materials and facilities as may be required, and shall provide convenient access to all parts of the work for the purpose of observations to check compliance with the plans and specifications.

B. The Engineer will test the LFCF density, strength, and foam density.
   1. Cast Density
      a. During placement of the initial batches, check the density and adjust the mix as required to obtain the specified cast density at the point of placement.
      b. At hourly intervals during placing, monitor the density and adjust as necessary to maintain the specified cast density.
   2. Strength
      a. The Contractor shall supply a sufficient quantity of 3-inch diameter by 6-inch high cylinder molds for use by the Engineer. For quantitative purposes, the Contractor shall assume that four test cylinders will be taken for each 70 cubic yards of LFCF placed and a minimum of two sets of four cylinders each per day.

         The Contractor shall supply a protective container, which will prevent damage and loss of moisture to the test specimens. The protective container shall be located on the site in an area approved by the Engineer.

         Strength determination shall be the responsibility of the Engineer. All cylinders will be molded, cured, handled, and tested by the Engineer. The Contractor shall provide molds and a curing environment conforming to the requirements of ASTM C495. The anticipated sequence of testing will be at 2, 7, 14, and 28 days.

3. Foam Density
   a. Foam density shall be checked twice per day per operating unit for compliance with the approved submitted mix design.

COMPENSATION

4.01 METHOD OF MEASUREMENT

A. Measurement for Lightweight Foam Concrete will be by the cubic yard, in place.

B. No separate measurement will be made for sheet membrane, waterproofing membrane, grading and compacting, or welded wire fabric.

4.02 BASIS OF PAYMENT

A. Payment for the work of this Section will be at the Contract price per cubic yard for Lightweight Foam Concrete Fill, which shall be full compensation for all efforts necessary to perform the work of this Section.

4.03 PAYMENT ITEM

Lightweight Foam Concrete Fill	Cubic Yard

Wood Fiber - Washington DOT

WOOD FIBER IN-PLACE

Where shown in the plans or where directed by the Engineer, the Contractor shall furnish, load, haul, place, and compact wood fiber borrow in place.

Materials

The wood fiber borrow shall consist of 100 percent wood fibers, such as sawdust, hog fuel, or wood chips. No composition wood products, such as particle or chipboard, pressed hard board, or presto-log fragments, shall be used in this embankment. Maximum size shall be 150 mm (6 in.) in the greatest dimension. Sufficient smaller-sized material shall be used to produce a uniformly dense fill. Cedar sawdust borrow will not be allowed.

Construction

The wood fiber borrow embankments may be constructed by dumping from trucks or by any other methods approved by the Engineer. Sawdust borrow shall be placed in lifts a maximum of 0.3 m (1 ft.) in depth of un-compacted material.

Compaction shall be obtained by covering the entire surface of each lift with a minimum of two passes with a D8-Caterpillar tractor or other similar compaction units as approved by the Engineer. Hauling units shall be routed over the entire fill for additional compaction.

Measurement

Wood fiber borrow in place will be measured by the cubic meter of neat line volume in place.

Payment

The unit contract price per cubic meter for "Sawdust Borrow in Place" shall be full compensation for furnishing all labor, tools, equipment, and materials necessary or incidental to complete the work as specified, including loading, hauling, placing, and compacting.

Shredded Tire - Model Specification

This model specification has been developed by Dr. D.H. Humphrey and is based on construction considerations outlined in ASTM D6270-98.

LIGHTWEIGHT TIRE SHRED FILL

Description
Lightweight tire shred fill shall consist of furnishing, placing, and compacting tire shred fill to the lines designated on the plans or specified or directed by the Engineer, including the preparation of the areas upon which tire shred fill is to be placed.

Tire shreds
The tire shreds shall be made from scrap tires, which shall be shredded into the sizes specified herein. They shall be produced by a shearing process. Tire shreds produced by a hammer mill will not be allowed. The tire shreds shall be free of any contaminates, such as oil, grease, gasoline, diesel fuel, etc., that could leach into the ground water or create a fire hazard. In no case shall the tire shreds contain the remains of tires that have been subjected to a fire. The tire shreds shall be free from fragments of wood, wood chips, crumb rubber and other fibrous organic matter.

The tire shreds shall have less than 1 percent (by weight) of metal fragments that are not at least partially encased in rubber. Metal fragments that are partially encased in rubber shall protrude no more than 25 mm (1 in.) from the cut edge of the tire shred on 75% of the pieces and no more than 50 mm (2 in.) on 100% of the pieces.

At least one sidewall shall be severed from the tread of each tire. A minimum of 90% (by weight) of the shreds shall have a maximum dimension, measured in any direction, of 300 mm (1 ft.) and 100% of the shreds (by weight) shall have a maximum dimension of 450 mm (17.7 in.). The tire shreds shall meet the following grading requirements:

Sieve Sizes	Percent Passing
300 mm	100
200 mm	75-100
38.1 mm	0-25
4.75 mm	0-1

The gradation shall be measured in accordance with AASHTO T-27, "Standard Method for Sieve Analysis of Fine and Coarse Aggregate," except that the minimum sample size shall be 30 pounds (12 kilograms).

Placing

The subgrade to receive tire shred material, immediately prior to spreading, shall conform to the compaction and elevation tolerance specified for the material involved and shall be free of loose or extraneous material. Tire shreds shall not be placed on soil containing organic matter.

The tire shred course shall be enclosed in a layer of geotextile as shown on the plans. The geotextile shall meet the requirements *(fill in appropriate requirement for high survivability)*. The seams formed by adjacent strips of filter fabric shall be overlapped a minimum of 450 mm (17.7 in.). Payment for the geotextile shall be considered incidental to construction of the tire shred fill.

Tire shred placement shall not be performed when material is frozen or a blanket of snow prevents proper compaction.

Compacting

The tire shred course shall be constructed in layers of uniform thickness and each layer shall be compacted in accordance with the requirements specified in this Section. The maximum compacted thickness of any tire shred layer shall not exceed 300 mm (1 ft.). Each layer of tire shreds shall be placed over the full width of the section. The tire shreds shall be spread with track mounted bulldozers, rubber tired motor graders, backhoes, or other equipment as needed to obtain a uniform layer thickness. The tire shreds as spread shall be well mixed with no pockets of either fine or coarse tire shreds. Segregation of large or fine particles will not be allowed.

Each lift of tire shreds shall be compacted with six passes of a vibratory smooth drum roller with a minimum static weight of 10 tonnes.

If the top of any layer becomes contaminated by addition of foreign material, including, but not limited to, soil, organic matter, oil, grease, gasoline, or diesel fuel, the contaminated material shall be removed and replaced with the specified material at no additional cost.

The completed side slopes and surface of the tire shred course shall be brought to a condition of uniform stability and compaction. To compensate for settlement of the tire shreds caused by the weight of the overlying soil, the top surface of the tire shred fill shall be overbuilt as shown on the plans. A tolerance of 75 mm (3 in.) above or 75 mm (3 in.) below the required grade and cross section will be allowed.

Sampling

The Engineer shall take samples during startup of tire shred production, and then random samples will be taken one to two times per week during production. Additional samples may be taken if deemed necessary by the Engineer. For the purpose of obtaining samples, the Contractor shall provide access to the discharge conveyor and the tire shred stockpiles, as requested by the Engineer. Tire shreds that do not meet the specifications shall be reprocessed to produce tire shreds meeting the specifications or shall be removed by the Contractor utilizing an approved disposal method at no additional cost.

Low Permeability Cover

The side slopes and top of the tire shred layer shall be covered by 0.5 m (1.6 ft.) thick low permeability soil cover, unless otherwise shown on the plans or directed by the Engineer. The soil shall have an AASHTO classification of A-4, A-5, A-6, or A-7. The low permeability cover shall be placed and compacted as specified *(fill in appropriate reference)*. The top of the low permeability cover shall be sloped to drain with a minimum slope of 1 percent. Construction of the low permeability cover shall be paid in accordance with Section *(fill in appropriate reference)*.

Waiting Period

After bringing the overlying aggregate base to the approximate final grade, the Contractor shall allow a period of at least 30[1] days for settlement of the tire shred layer prior to final grading and paving.

[1] Use 60 days if the tire shred fill cannot be uniformly tapered from the full thickness to zero over a horizontal distance of at least two times the thickness of the tire shred fill.

Instrumentation

(Add this section if needed)

Measurement

The tire shred course placed in accordance with the widths and thickness shown on the plans and compacted as specified will be measured by the tonne.

Payment

Lightweight tire shred fill will be paid for by the tonne, which price shall be considered full compensation for all labor, materials, and equipment required to furnish an acceptable tire shred course and associated geotextile layer.

Fly Ash Delaware - DOT Specification

Description

This work shall consist of furnishing and constructing a drainage foundation with underdrains and outlets and a Class F fly ash embankment with soil slopes and cover at the locations and to the depths, lines, and grades shown on the plans. The acquisition and delivery of Class F fly ash to the site shall be the responsibility of the Contractor.

Materials

Class F fly ash shall be supplied from a source approved by the Department. The ash shall not contain bottom ash with pyrites and shall not be frozen. Ash from boilers fired with both coal and petroleum coke will not be approved. Mixtures of fly ash and bottom ash, or fly ash and soil, will not be permitted. Material properties and laboratory analysis of ash typical of the source shall be submitted to the Department at least 30 calendar days prior to use for consideration of approval. Test data shall include characteristics of the ash leachate as determined by the EPA EP Toxicity Test Procedure.

Drainage Foundation

A drainage foundation, as shown on the plans, shall be constructed prior to placement of the fly ash embankment. It shall consist of 0.3 m (1 ft.) of washed sand and conform to Section 804, Fine Aggregate, and a minimum of 0.6 m (2 ft.) of Borrow Type A, meeting the requirements of Section 209. In addition, the drainage foundation longitudinal underdrain system and outlet system shall be constructed in accordance with the details and at locations shown on the plans. There shall be no separate payment for furnishing, placing, and compacting the drainage foundation, underdrain and outlet systems, and all costs shall be considered incidental to this item.

Hauling

Fly ash shall be delivered to the project, conditioned by the addition of water in covered trucks. Excessively wet or dry ash arriving at the project or ash delivered in trucks without covers properly in place will be immediately rejected. To prevent the occurrence of highway spillage, the Contractor shall not overfill trucks. Coordination of fly ash supply, including adequate daily volumes to maintain project progress, trucking requirements, and ash moisture content control shall be the responsibility of the Contractor.

Construction Methods

The fly ash embankment shall be constructed upon a 1 m (3.3 ft.) thick foundation consisting of 0.3 m (1 ft.) of washed sand placed in a single layer and compacted with 2 passes of a vibrating roller, and a minimum of 0.6 m (2 ft.) of compacted borrow Type A, meeting the

requirements of Section 209. The fly ash shall constitute the embankment core and be contained within dikes formed of compacted Borrow Type F, Section 209, at the embankment slopes as shown on the plans. The fly ash shall be delivered to the project with a moisture content within 4 percent of optimum moisture but not exceeding optimum moisture by greater than 1 percent. The fly ash shall be spread out, tracked in with a dozer and compacted immediately after it has been deposited. Stockpiling of the fly ash in the embankment areas or in other areas outside of its source will not be permitted. The placement of fly ash on wet, unstable or frozen materials will not be permitted. The fly ash shall be placed in a level uniform cross section not exceeding 200 mm (8 in.) in depth loose measurement. It shall be deposited and spread parallel to the centerline and the layers shall extend the full width of the embankment. The loose lift shall be tracked in with a dozer prior to compaction. A water truck shall be available to add water prior to compaction if needed, in order to obtain the desired range of moisture.

Compaction shall be obtained with a 9 Mg, self-propelled steel-wheel vibratory roller. The compaction or rolling shall start at the edges and progress toward the center of the embankment. The first pass of the roller shall be with the vibrator off. Compaction shall continue until each layer is thoroughly and uniformly compacted to the full width of the embankment. In order to eliminate laminations between layers, the final pass with the roller shall be made in reverse with the vibratory motion off. If necessary the surface shall be disked in order to eliminate laminations.

The fly ash core shall be brought to within 150 mm (6 in.)of the level of the adjacent dikes at the end of each day and graded to provide surface drainage to prevent ponding and saturation of the fill due to rain. The contractor shall construct openings in the dike and provide Flexible Pipe Slope Drains at locations directed by the Engineer. The furnishing and installing of the flexible pipe slope drains shall be paid for under Item 740502 Erosion Control, Slope Drain and they shall be installed as per Item 740502.

There shall be sufficient equipment of the proper type and weight provided to do the work of grading, leveling, and compacting promptly after depositing the material. When this equipment is inadequate for the rate of compacting, the rate of excavation or placing of embankment shall be reduced to a rate not to exceed the capacity of the grading and compacting equipment.

The Contractor shall establish grade stakes in the field and submit for approval by the Engineer the final grades of the top of the drainage foundation layers (top of Borrow, Type A and Section 804 fine aggregate) and the top of the fly ash prior to placement of the subsequent layer of material. The Engineer shall allow a minimum of ten (10) working days

for review. Adjustments to the final grades as directed by the Engineer shall be performed by the Contractor with all costs incidental to this item.

Dusting will not be permitted and will be controlled throughout the work day, and when directed by the Engineer, through the application of a water spray or chemical wetting agent, such as Coherex, Acrylic Drl-Ms, Hercobind SP-28, or an approved equal. Water spray shall be such that it will not cause excessive wetting of the fly ash.

Compaction Equipment

Approved rollers or compactors shall attain compaction. The use of other suitable compaction equipment may be approved for work under this Section provided such equipment achieves the compaction requirements of these specifications. However, heavy vibratory compaction equipment will achieve the required in-place densities over a wider range of moistures than lighter vibratory and static rollers.

Density and Moisture Control

The determination of compliance with the field compaction and moisture content requirements specified herein shall be in accordance with the following AASHTO test methods:

AASHTO T 99, Method C - Determination of Maximum Density and Optimum Moisture Content.
AASHTO T 239 Density of Soil In-Place by Nuclear Methods
AASHTO T 272 Family of Curves: One-Point Method

The moisture content of the fly ash at the time of compaction shall be within 4 percent of optimum moisture, but not greater than 1 percent in excess of optimum moisture, as determined by AASHTO T 99, Method C. If the fly ash moisture is not within this moisture range, the ash shall be either moistened or dried, as directed by the Engineer, and thoroughly mixed before compaction.

Each fly ash layer shall be uniformly compacted to 95 percent or more of the laboratory maximum standard proctor density obtained on ash representative of the material in use. A series of field density tests will be made by the Engineer during the compaction operation to verify compaction quality. Nuclear density gauges will be used in conjunction with the family of curves method to determine the compaction level achieved by the Contractor's equipment and placement method. Equipment or placement methods shall be modified as necessary if the specified compaction level is not achieved. At the discretion of the Engineer, the Contractor may be directed to utilize a control strip to develop the number of

roller passes necessary to achieve the minimum 95 percent compaction level for the lift thickness and moisture content of ash being placed.

Method of Measurement

Fly-ash embankment will be measured by computing a template volume in cubic yards from the cross sections and plans. Average end area will be the method of computation. No adjustment factor will be applied to the quantity computed by plan template volume. Included in the measurement and computation will be the drainage foundation required just below the fly ash core.

Basis of Payment

This item will be paid at the contract unit price bid per cubic yard for "fly ash," which price and payment shall include furnishing, placing, and compacting the drainage foundation, longitudinal underdrain system, outlet system, and fly ash embankment core to the lines and grades shown on the plans and in accordance with this special provision. Also included are all costs for wetting, drying, and applying the chemical wetting agent. All water application costs for dust control during fly ash embankment placement, establishment of appropriate grades, and adjustments to grades are incidental to this item.

Technical Summary # 3:

VIBRO-COMPACTION

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CHAPTER 1
DESCRIPTION AND HISTORY

1.1 DESCRIPTION

Vibro-compaction is a ground improvement technique that uses specially designed probe-type, depth vibrators for in-situ densification of loose sands and gravels, as shown in Figure 1. Originally called Vibroflotation, the process was accomplished with water jetting, hence the name.[1] 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 technical summary 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.[2]

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 capacity, permitting shallow foundation construction
- reduced foundation settlements
- increased resistance to liquefaction
- increased shear strength
- reduced permeability
- filling of voids in treated areas

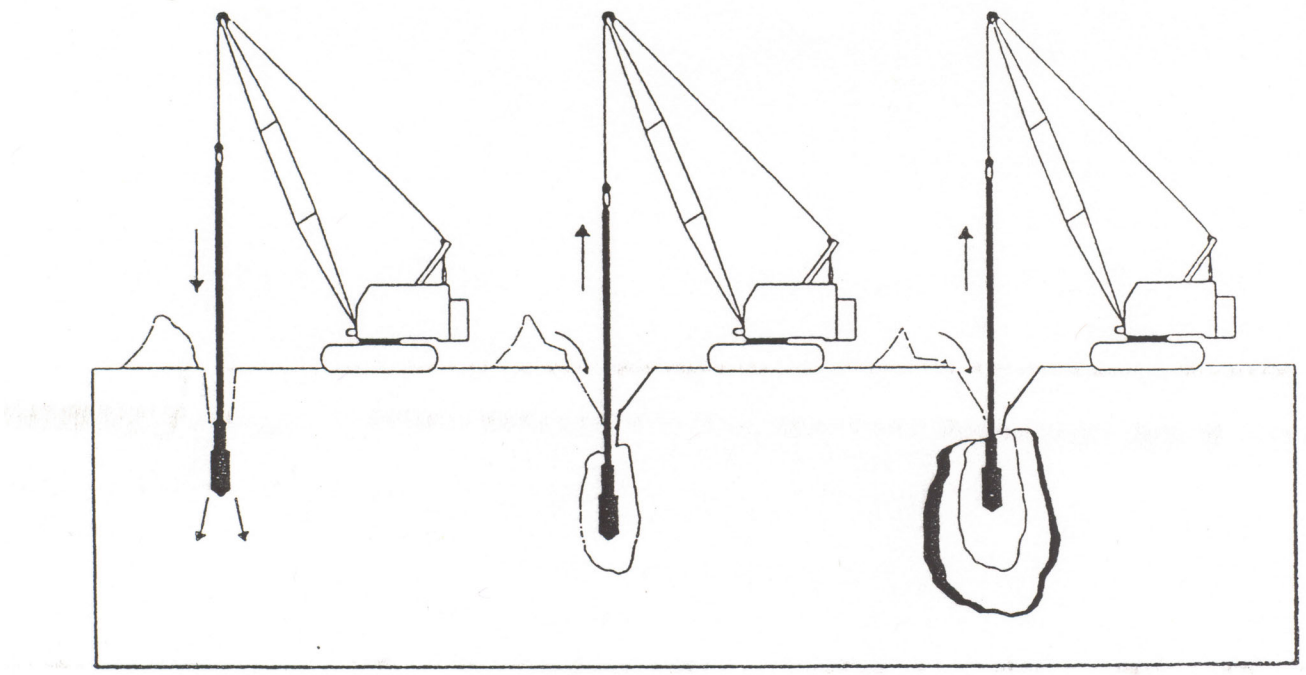


Figure 1. The Vibro-Compaction Process.

As with any ground improvement 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 FOCUS AND SCOPE

As it is a mature, well-established method of densifying loose, granular soils, little new information has been written about vibro-compaction in recent years. The intent of this technical summary is to present the state of the practice in utilizing depth vibrators to densify granular soils. A historical overview will be presented followed by applications, advantages and disadvantages/limitations, feasibility evaluations, equipment, required materials, design concepts, specifications, inspection control, and verification. Case histories, cost data, and references are also included.

1.3 HISTORICAL OVERVIEW

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 20 m (65 ft.). 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. In order 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. They 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.[3,4]

The Great Alaska and Nigata 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.

1.4 SUGGESTED READING

Since the early 1950s in the United States, many technical articles have been written on various aspects of vibro-compaction. The following have been selected as recommended reading:

Vibro-Compaction

- "Vibroflotation Compaction of Cohesionless Soils" [5]
- "The Development of Compaction Methods with Vibrators from 1976 to 1982" [6]
- "Vibratory Deep Compaction of Underwater Fill" [7]
- "Wando Terminal Ground Improvement Program" [8]
- "Vibroflotation in Reclamations in Hong Kong" [24]

General Ground Improvement

- "Soil Improvement: State-of-the-Art Report" [9]
- "Soil Improvement – A Ten Year Update" [3]
- "In-Situ Deep Soil Improvement" [10]
- "Soil Improvement for Earthquake Hazard Mitigation" [11]
- "Ground Improvement, Ground Reinforcement, Ground Treatment: Developments 1987-1997" [4]

CHAPTER 2
DESIGN CONSIDERATIONS

Vibro-compaction can be used to achieve a number of design objectives, as listed in Chapter 1. This chapter discusses applications for transportation facilities, as well as advantages, disadvantages, limitations of the system, and feasibility. Alternative site improvement methods are also addressed.

2.1 APPLICATIONS

This technical summary focuses on the use of vibro-compaction as a solution to problems related to transportation projects. Well over 2,000 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

2.2 ADVANTAGES AND DISADVANTAGES/LIMITATIONS

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 can also be a cost-effective alternative to removal and replacement of poor load-bearing soils. The use of vibro-compaction allows the maximum improvement of granular soils to depths exceeding 50 m (165 ft.). The vibro-compaction system is effective both above and below the natural water level.

Disadvantages/Limitations

The major 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 12-15 percent silt OR more than 2 percent clay. The maximum depth of 50 m (165 ft.) may be considered a disadvantage, but there are very few construction projects that will require densification to a greater depth.

Like all ground improvement 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.

2.3 FEASIBILITY EVALUATIONS

a. In-Situ Soil Gradations

The suitability of a soil for vibro-compaction methods depends mainly on its grain size distribution, as shown in Figure 2. Soils with grain size distribution curves lying entirely on the coarse side of zone B are readily compacted with depth vibrators. 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 the companion technical summary "Ground Improvement by Stone Columns."

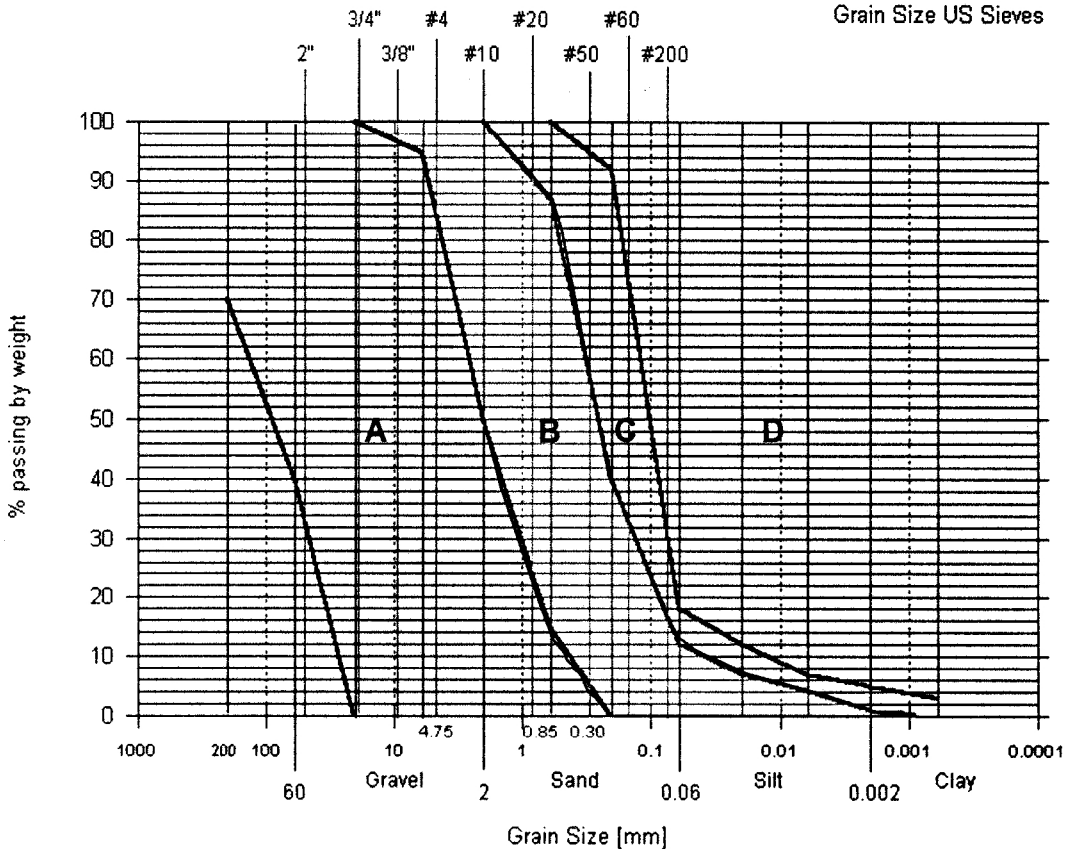


Figure 2. Range of Soil Types Treated by Vibro-Compaction.

For non-cohesive 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. [(2)]

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.

b. 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.

2.4 ALTERNATIVE DEEP COMPACTION METHODS

A number of alternative methods of compacting ground in-situ also exist. Some of these techniques are as follows:

Compaction or Displacement Piles

Compaction or displacement piles are any driven piles that displace the ground to obtain densification. Vibro-compaction is generally more economical than the use of compaction piles. The spacing for compaction piles to achieve comparable results is usually less than or equal to vibro-compaction because they improve a smaller soil volume per penetration. However, compaction piles are usually more expensive on a unit price basis. In cases where piles must be driven at close centers for load-carrying purposes, the compaction achieved may be sufficient to eliminate the need for vibro-compaction.

Frankipaction

The Frankipaction system is essentially an adaptation of the Frankipile to achieve compaction. The Frankipile 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.

Generally, for densification, vibro-compaction methods are more economical than Frankipaction.

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 (>30 m (100 ft.)). As with dynamic compaction (described below) the technique has the 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.[14]

The worldwide experience in blast densification both above and below the groundwater table and the design framework to implement projects has been summarized by Narin Van Court and Mitchell in Explosive Engineering.[15]

Dynamic Compaction

The system of dynamic compaction is the most economically competitive system to vibro-compaction. Experience has indicated that dynamic compaction is limited to depths of 9-12 m (30-40 ft.), regardless of how much energy is applied. A high water table will significantly increase dynamic compaction costs. Vibro-compaction, however, can be accomplished to great depths (typically to 30 m (100 ft.) and 50 m (165 ft.) maximum) and is not affected by the water table. Vibro-compaction methods can be used as close as ~2.0 m (6.6 ft.) from existing structures, whereas dynamic compaction is not recommended within 30 m (98 ft.) of existing structures due to the potential for structural damage from vibrations generated by the impact of the weight.

The common applications of this technique are described in a companion technical summary "Dynamic Compaction."

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 companion technical summary, "Grouting." The use of compaction grouting as a densification tool is relatively new. Although vibro-compaction and 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.

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 0.8 m (2.6 ft.). The vibro-compaction method transmits a horizontal vibration over a distance of up to 4 m (13 ft.). 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.

CHAPTER 3
CONSTRUCTION AND MATERIALS

The vibro-compaction process uses crane-mounted depth vibrators and appropriate backfill material. This chapter discusses construction equipment and the suitability of backfill material.

3.1 EQUIPMENT

The equipment used to achieve the necessary densification are high-powered, probe-type vibrators ranging from 0.3 m to 0.40 m (12 in. to 16 in.) in diameter and 3 m to 4.5 m (10 ft. to 15 ft.) in length, as shown in Figure 3 and Table 1. 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, drives the rotating shaft. To drive this assembly, 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 150-450 kN (33,750-100,000 lbs) 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.

Table 1 describes selected available vibrators and some of their operating characteristics.

Figure 3. High-Powered, Probe-Type Vibrator Utilized in Vibro-Compaction.

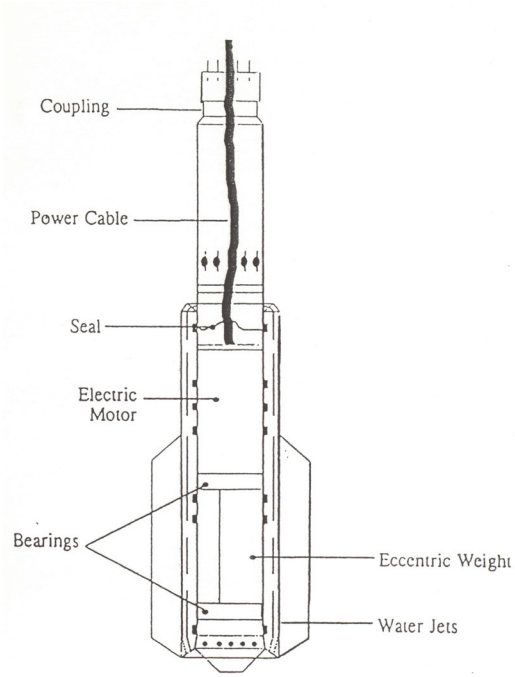


Figure 4. Cross-Section of a Typical Vibrator.

Table 1. Specifications of several vibrators.[23]

Manufacturer:	Bauer	Bauer	Keller	Keller	Keller	Keller	Vibro	Vibro
Name	TR13	TR85	M	S	A	L	V23	V32
Length m	3.13	4.2	3.3	3.0	4.35	3.1	3.57	3.57
Dia. mm	300	420	290	400	290	320	350	350
Weight kg	1000	2090	1600	2450	1900	1815	2200	2200
Motor kW	105	210	50	120	50	100	130	130
Speed rpm	3250	1800	3000	1800	2000	3600	1800	1800
Ampl. mm	6	22	7.2	18	13.8	5.3	23	32
Dynamic Force kN	150	330	150	280	160	201	300	450
Note: For see table at front of manual for SI conversions.								

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 5), a specially built hydraulic crawler crane, or a crane that is mounted on a barge (Figure 6), depending upon the site conditions.

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.

Figure 5. Vibrator Suspended from a Conventional Crane.

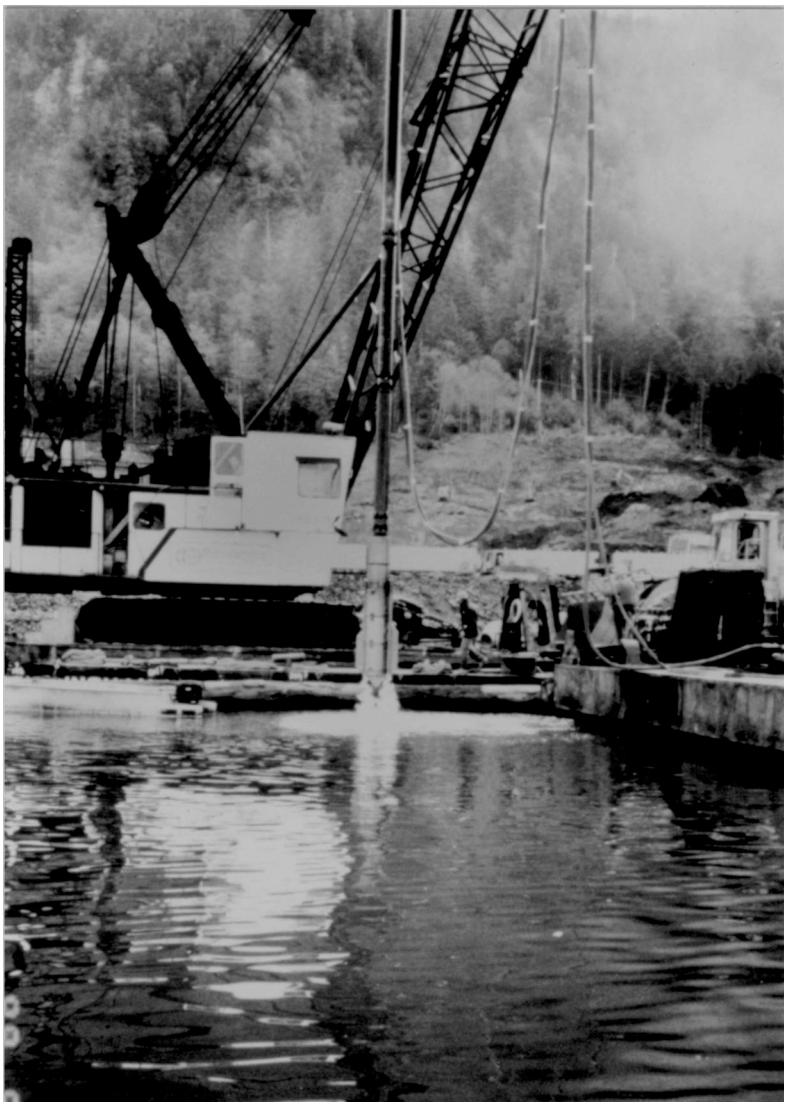


Figure 6. Vibrator Suspended from a Barge-Mounted Crane.

3.2 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.[5] 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:

$$Suitability\ No. = 1.7\sqrt{\frac{3}{(D_{50})^2} + \frac{1}{(D_{20})^2} + \frac{1}{(D_{10})^2}} \quad (1)$$

where D_{50}, D_{20}, and D_{10} are the grain size diameters, in millimeters, at 50 percent, 20 percent, and 10 percent passing. Table 2 explains the qualitative categories of backfill utilizing this rating system more precisely. 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 0.074 mm (#200) sieve, and containing no clay.

Table 2. Backfill evaluation criteria.

Suitability Number	0-10	10-20	20-30	30-40	>50
Rating	Excellent	Good	Fair	Poor	Unsuitable

CHAPTER 4
DESIGN CONCEPTS

Design of a vibro-compaction program requires initially a 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 resistance requirement.

4.1 PROBLEM DEFINITION

The solution of all design problems can be resolved by following these steps:

1) Identify the problem.
2) Determine the potential technical solutions.
3) Determine the most economical solution.
4) Prepare appropriate plans and specifications.

If loose granular soil is identified as the problem, then densification by vibro-compaction will be a potential technical solution, especially if the loose deposit is deeper than 10 m (33 ft.) as measured from the surface. Table 3 indicates 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.

If vibro-compaction is selected as the improvement method, the following parameters must be determined:

- the gradation of the in-situ soils, including silt and clay content
- the existing relative density, or looseness, of the in-situ soils
- the required density improvement necessary to solve the project's requirements and, once determined, whether this improvement is feasible

Table 3. Penetration resistance and sand properties.

	Very Loose	Loose	Medium Dense	Dense	Very Dense
SPT N-value (blows/0.3m)*	<4	4-10	10-30	30-35	>50
CPT cone resistance (kg/cm^2)	<50	50-100	100-150	150-200	>200
Equivalent Relative Density (%)*	<15	15-35	35-65	65-85	85-100
Dry Unit Weight (kN/m^3)	<14	14-16	16-18	18-20	>20
Friction Angle	<30	30-32	32-35	35-38	>38
Cyclic Stress Ratio[12] Causing Liquefaction	>0.04	>0.04-0.12	>0.12-0.33	>0.33-0.40	
Shear Wave Velocity[24] m/s	<120	120-160	160-200	200-225	>225

NOTES:

** Normally consolidated sand.*

See table at front of manual for SI conversions.

4.2 PRELIMINARY DESIGN

The significant engineering properties of a granular soil – compressibility, shear resistance, permeability, resistance to dynamic loading – is 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:

$$D_r = \frac{\gamma_n - \gamma_l}{\gamma_d - \gamma_l} \chi \frac{\gamma_d}{\gamma_n} \chi 100\% \quad (2)$$

where: γ_n is the dry density of the soil in its natural state,
γ_l is the dry density of the soil in its loosest state,
γ_d is the dry density of the soil in its densest state.

In this calculation, γ_l and γ_d should be determined in accordance with current ASTM procedures.

High relative density corresponds to high bearing capacity 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 coefficient of compressibility is increased, and consequently settlements are greatly reduced.

In addition to soil gradation, the area influenced (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 1, vibrator characteristics vary widely.

Figure 7a shows the approximate relationships between relative density, soil type, and treatment area for a specific vibrator. It is unlikely, even for heavy loading, that it will be necessary to achieve a relative density above 85 percent.

Figure 7b is a chart useful in estimating the probable level of improvement that can be obtained by vibro-compaction. It is based on the lower bound soil gradation (silty sand) indicated in Figure 7a. Similar charts can be developed for coarser granular soils from Figure 7a.

The project designer is responsible for determining 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
- 70-75 percent relative density for column footings, bridge footings
- 80 percent relative density for machinery and mat foundations

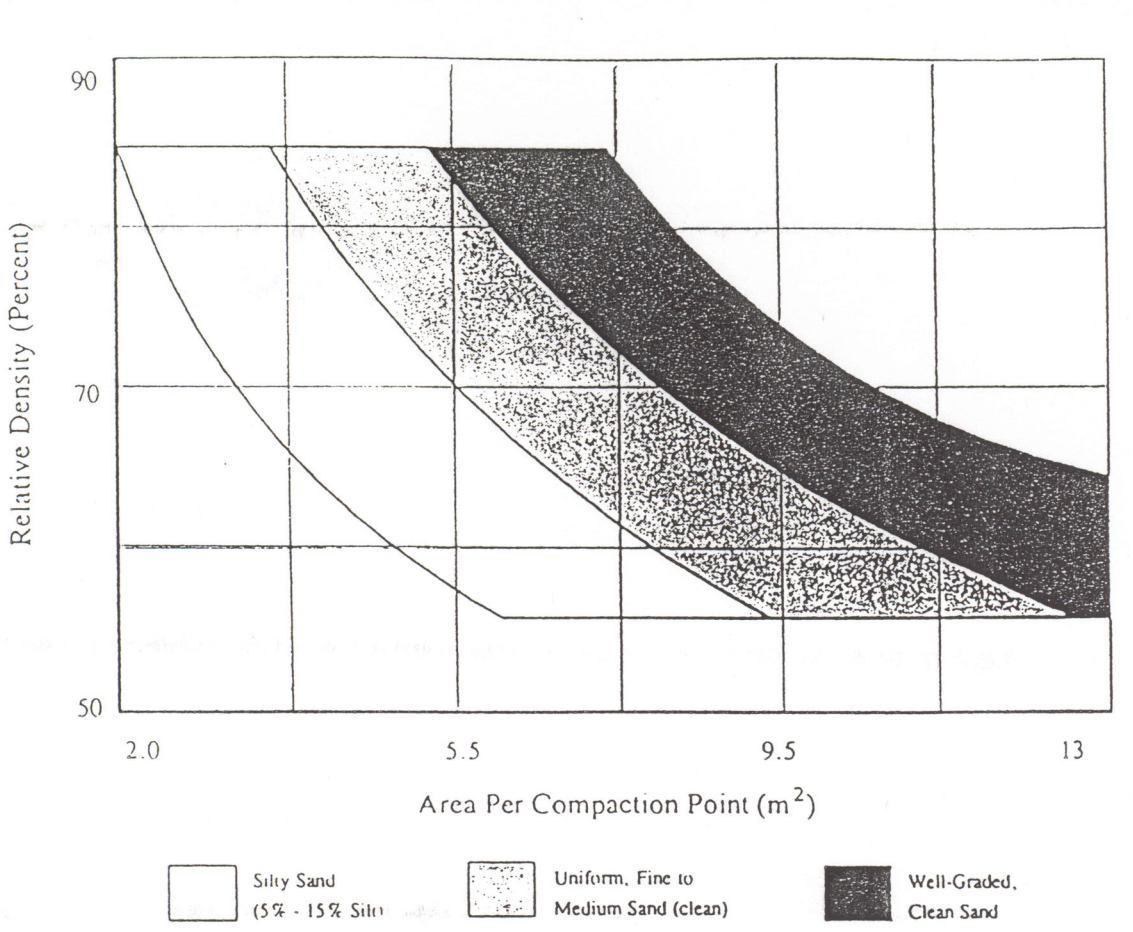


Figure 7a. Approximate Variation of Relative Density with Tributary Area.

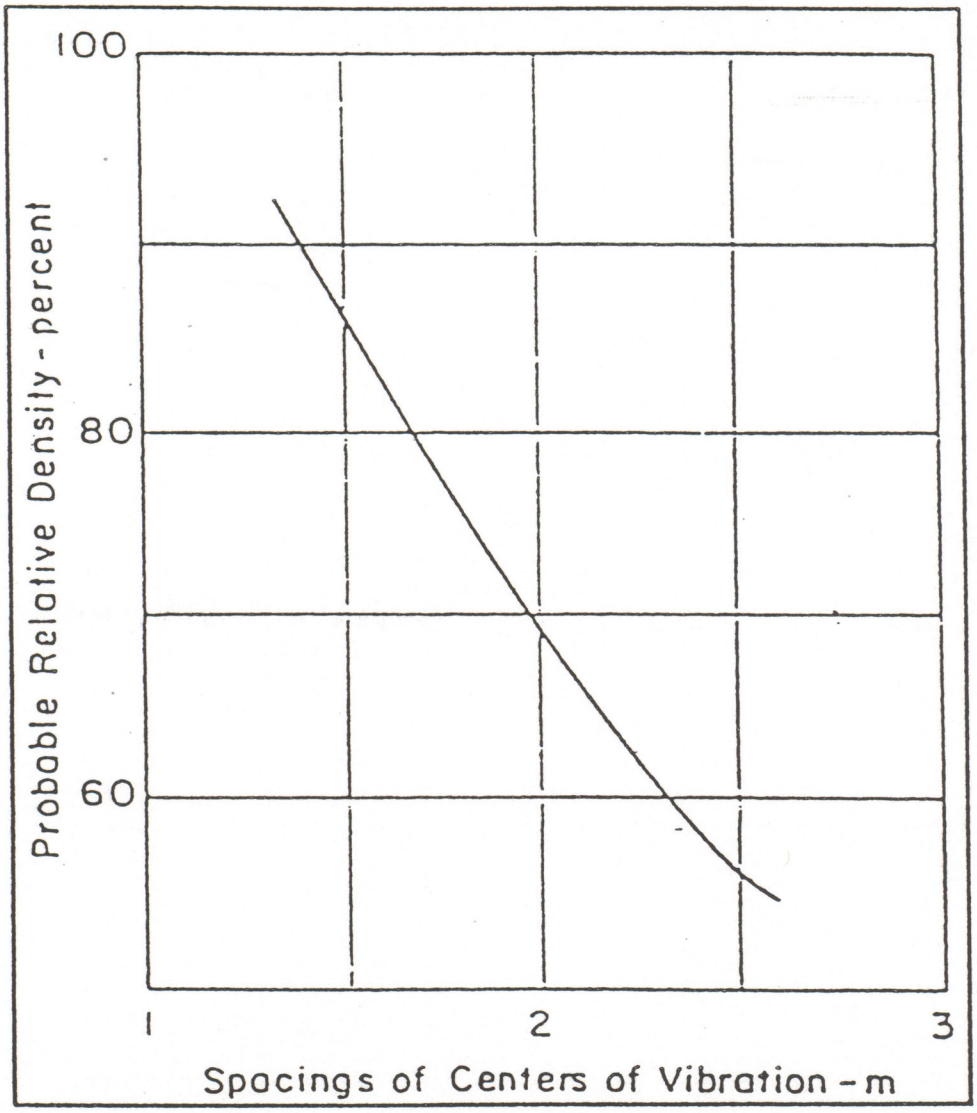


Figure 7b. Relative Density vs. Probe Spacing (for Silty Sands).

4.3 PRELIMINARY DESIGN FOR LIQUEFACTION MITIGATION

The identification of liquefiable soils is beyond the scope of this Technical Summary, 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 provided the following recommendations: [16]

- 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.

For non-critical structures and ground accelerations of less than 0.5g, the analysis involves confirming that the factor of safety against liquefaction is adequate.[16,17] Where densification is deemed necessary, the "simplified method" relationships are used to determine the required SPT $(N_1)_{60}$ or CPT q_{cl} values, which with the cyclic stress ratio (CRS) for the design earthquake is used to compute the liquefaction safety factor.[16,17]

The cyclic stress ratio for the design earthquake at any depth can be expressed as

$$CRS_{EQ} = 0.65\ [a_{max}/g]\ [\sigma_v/\sigma'_v]\ r_d \qquad (3)$$

where:

a_{max} = peak horizontal surface acceleration
g = acceleration of gravity
σ'_v = initial effective overburden stress
σ_v = total overburden stress at the same depth
r_d = shear stress reduction coefficient to adjust for the soil flexibility profile (reflects attenuation or amplification, that is dependent upon profile)

The reduction coefficient, which is simply a function of the magnitude of the design earthquake and the depth in question, can be obtained from Figure 8.

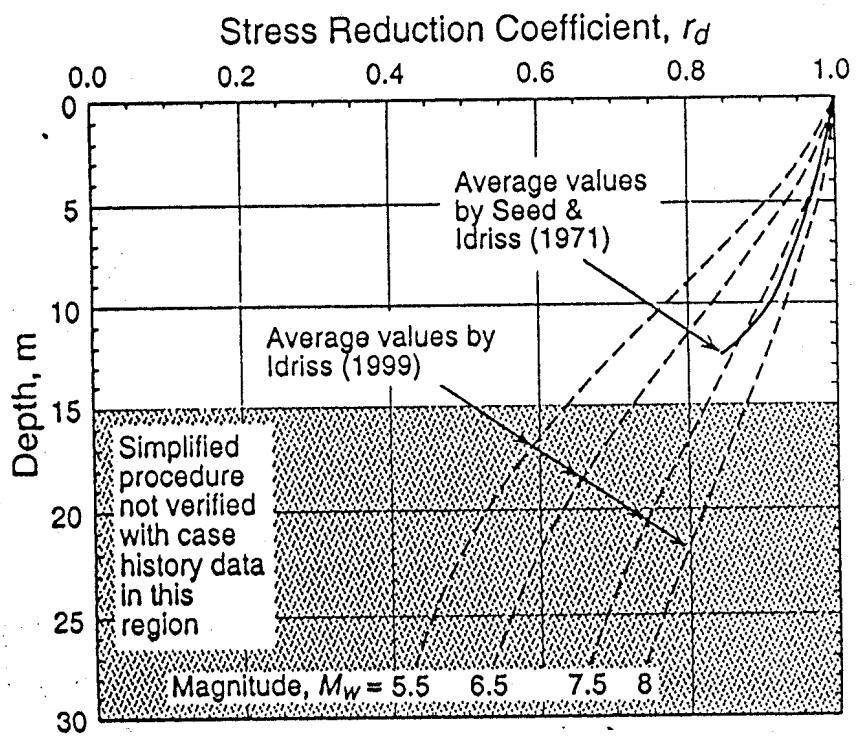


Figure 8. Shear Stress Reduction Factor.[13]

The standardized blow count $(N_1)_{60}$ can be evaluated by computing a correction factor as follows:[17]

$$(N_1)_{60} = C_N \cdot N_{60} \quad (4)$$

$$C_N = 9.79\ (1/\sigma'_v)^{1/2}$$

Where σ'_v is the vertical effective stress at the sampling point in kPa. N_{60} is the energy-corrected measured N value.

With knowledge of CRS and $(N_1)_{60}$ it can be determined from Figure 9 if liquefaction is likely. For more accurate determinations of the two variables, see Reference 17. Where liquefaction is indicated, the foundation can be improved by densification. For analyses using shear wave velocities, consult Reference 13.

Settlement attributed to earthquake shaking can be estimated using methods outlined in Reference 17. Seed indicates that the method outlined should only be used as a first approximation in evaluating the volume changes and settlements, and that the error associated with the estimation may be on the order of 50 percent.[22]

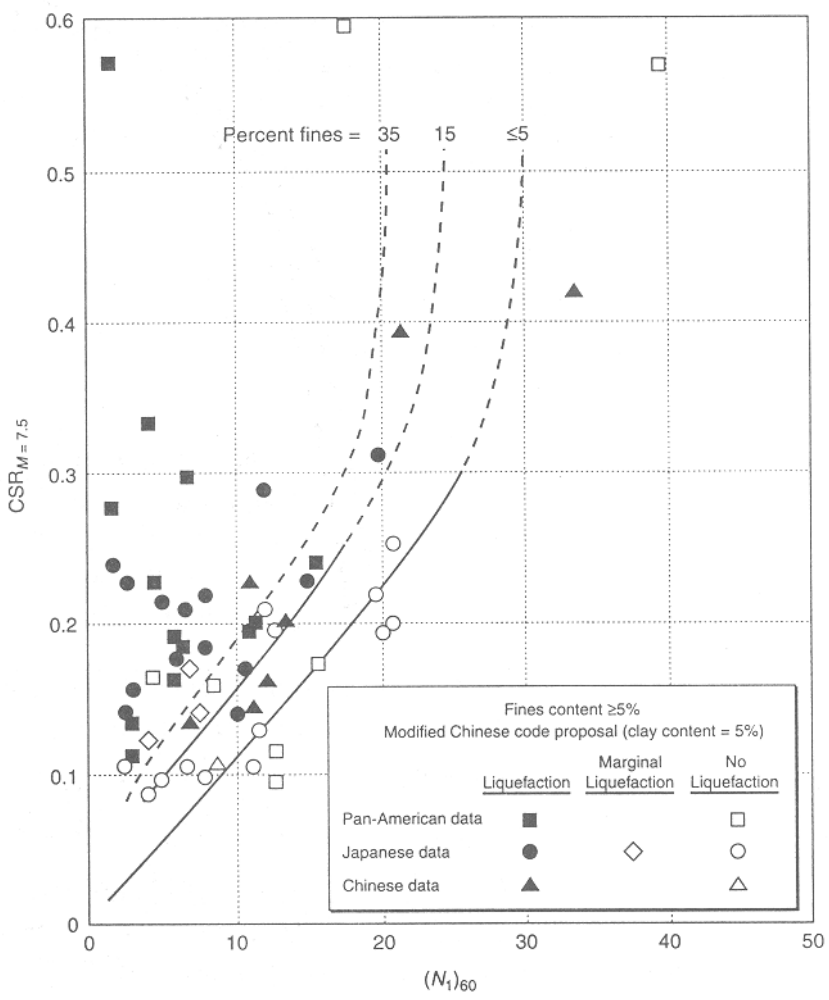


Figure 9. Relationship between Cyclic Stress Ratio (CSR) Causing Liquefaction and $(N_1)_{60}$ Values for Sands and M_w 7.5 Earthquakes.[17]

4.4 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.

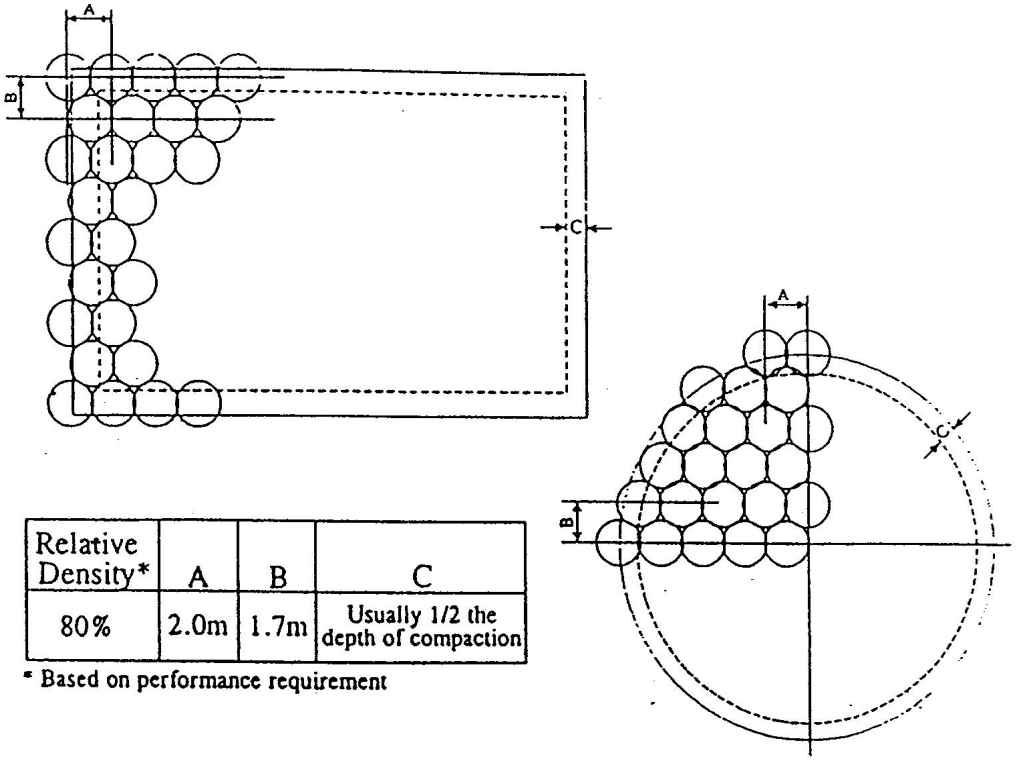
The area compaction point pattern also affects the densification. An equilateral triangular pattern is primarily used to compact large areas, since it is the most efficient pattern. The use of a square pattern instead of an equilateral triangular pattern requires 5-8 percent more points to achieve the same minimum densities in large areas.

Given the in-situ soil gradation and relative density required, the spacing of compaction points can be determined. Figures 10 and 11 show typical area patterns and spacing for 80 percent relative density requirements. The spacing of the vibro-compaction points would be wider for a lower relative density requirement.

4.5 PERFORMANCE REQUIREMENTS

Performance requirements, such as total or differential settlement, bearing capacity 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. Alternately, in-situ measurements of small strain shear wave velocity (V_{S1}) can be used to determine liquefaction potential and the required densification to preclude liquefaction. Typically, clean granular soils with small strain shear wave velocities in excess of 200 m/s (650 ft/s) will not liquefy, regardless of the cyclic shear stress ratio (CSR). To determine the potential for liquefaction at various shear wave velocities and cyclic shear stress, refer to Reference 13.



Relative Density*	A	B	C
80%	2.0m	1.7m	Usually 1/2 the depth of compaction

* Based on performance requirement

Figure 10. Typical Compaction Point Spacing for Area Layouts.

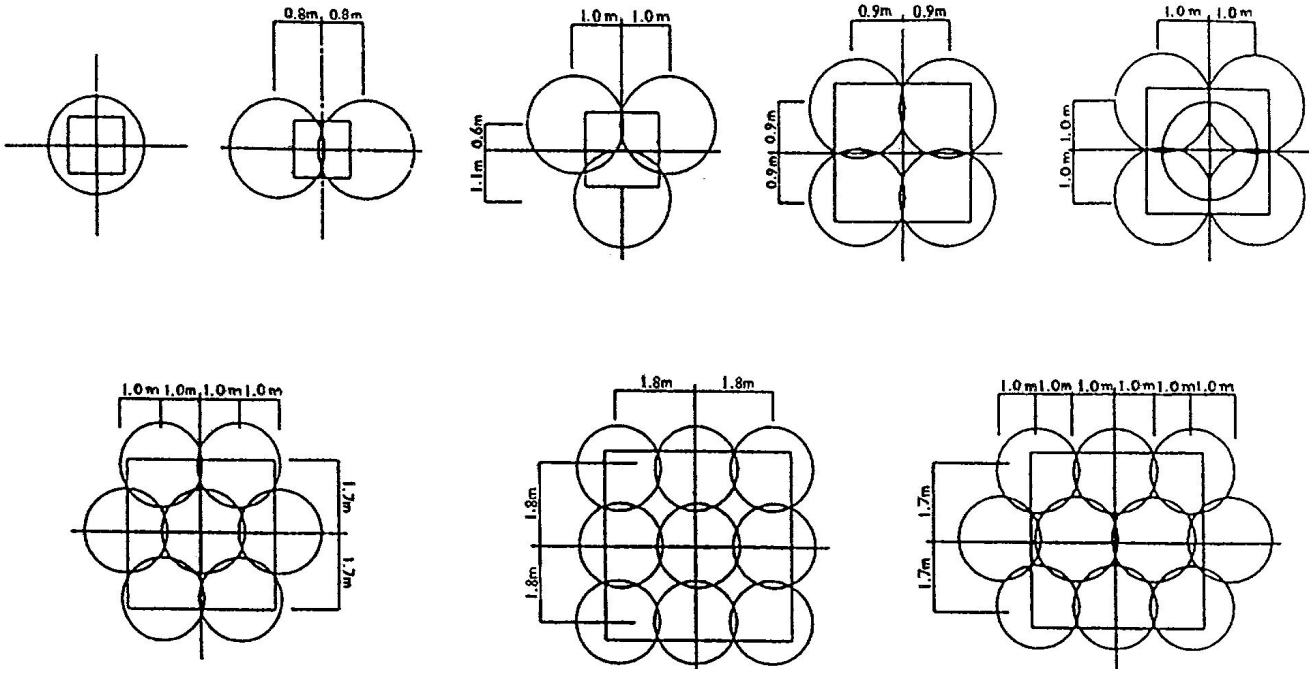


Figure 11. Typical Compaction Point Layouts for Column Footings.

CHAPTER 5
CONTRACTING METHODS, CONSTRUCTION SPECIFICATIONS, AND CONSTRUCTION CONTROL

5.1 CONTRACTING METHODS

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 chapter 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.[18, 24] A minimum wait of 5 days is recommended before performing verification testing, but a wait of about 10 days is preferrable. The effectiveness of soil improvement with time after treatment should be considered in performing tests and interpreting test results.

5.2 CONSTRUCTION SPECIFICATIONS

The following guide to the preparation of a performance-type specification can be used in the formulation of project-specific specifications. The format of this sample 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.

SAMPLE SPECIFICATION
FOR VIBRO-COMPACTION

A. SCOPE OF WORK

1. This section includes all supervision, labor, material, equipment, and related services necessary to perform all soil improvement by vibro-compaction as indicated on the drawings or specified herein.

2. The work includes subsurface soil improvement by vibro-compaction, and delivery and placement of all backfill necessary for the improvement process.

3. Soil improvement by the vibro-compaction method will be carried out beneath *(the column pads/strip footings/floor slab/general site area, embankment, bridge footings)* in order to *(increase the bearing capacity and/or reduce settlements and/or minimize the risk of liquefaction)*.

4. The Contractor performing the vibro-compaction shall have a minimum of 3-year experience and document 10 recent successful projects. The Superintendent should have a minimum 3-year experience record of performing on-site supervision of vibro-compaction work. Documentation substantiating this experience should be provided to the Engineer for review 30 days prior to beginning the vibro-compaction work.

5. Covered under another section is the removal of obstructions in proximity to the vibro-compaction work, and preparation of a level surface on which the vibro-compaction subcontractor can operate his equipment.

B. BACKFILL MATERIAL

1. It may be necessary to maintain site elevations during the vibro-compaction work. Backfill should consist of sand, sand and gravel, or crushed rock material with less than 10 percent by weight passing the No. 200 sieve, and containing no clay.

2. A sample or gradation of any backfill material to be used should be submitted to the Engineer for approval prior to commencement of work.

C. EQUIPMENT AND PROCEDURES

The Contractor should select specific equipment and construction procedures to achieve the specified criteria. However, the following general guidelines are identified:

1. The Contractor should use a down-hole vibrator capable of providing at least 100 kW of power and 150 kN (33,750 lbs.) of force. The vibrator shall be suspended by a track-mounted crawler crane and lowered by its own weight and vibration with the aid of jetting water at the nozzles and wash pipes.

2. After penetration to the treatment depth, water output at the bottom jets shall be minimized and controlled, and the vibrator should be slowly retrieved in 0.3-0.5 m (1-1.6 ft.) increments to allow backfill placement.

   The vibrator should re-penetrate each backfill increment into a recently treated depth interval to observe amperage build-up or increase. *(Amperage build-up and backfill quantities are contingent on the type of vibrator, type of backfill, in-situ soil conditions, and Contractor's procedure. Discussion between the Engineer and Contractor should be conducted prior to beginning the vibro-compaction, regarding individual equipment capabilities and expectations.)*

3. Upon completion of the soil improvement work, depressions in the compaction area shall be filled with soil/rock as specified in Section B, and compacted by vibratory roller equipment.

D. ACCEPTANCE CRITERIA

1. Provide densification of the underlying soils as necessary to achieve the following: (*The Engineer must choose applicable criteria.)*

   a. a relative density of ____ according to _____. *(A correlation such as that developed by either Gibbs and Holtz, Schmertmann or Seed is typically specified.)*

   b. an ultimate bearing capacity of ___ as determined by *(load test, SPT correlation)*.

   c. total settlement limited to ___ . *(Define load and time.)*

d. differential settlement limited to _____ over a distance of _____. *(Define load and time.)*

e. an average corrected "N" value of ___ for SPT testing. *(Define average for project)*

f. an average cone resistance value of ___ for CPT testing. *(Since Cone Penetrometer Testing does not provide a soil sample, some test boring data must be correlated with a soil type. The Campanella and Robertson correlation may be used for defining untreatable strata from the friction ratio and pore pressure data. Pre-construction correlations may also be performed to confirm these untreatable soils.)*

2. The work should consist of densification of the underlying soils to a depth of ____. *(The densification achieved and/or engineering properties of the subsoil mass after vibro-compaction is dependent on many variables. These include the densification technique employed, the depth of treatment, the type and quality of the equipment mobilized, the experience of the Contractor, the backfill material utilized, the procedure used by the Contractor, and the engineering and management support provided by the Contractor.)*

Typically, vibro-compaction in clean granular soil can achieve densification to the following levels:

SPT N_{60} Value	*15-40 blows / 0.3 m*
CPT Resistance	*75-20 MPa*
Relative Density	*65-85 percent*

E. TESTING AND INSPECTION

1. All testing to determine specification compliance will be provided by *(the State testing agency; an independent testing agency; or the Contractor)* and should consist of (standard penetration testing *(SPT ASTM D 1586);* static cone penetration testing *(CPT ASTM D 3441);* plate load tests *(ASTM D 1194);* etc. The same test method should be utilized both before and after the soil improvement work.

2. Provision should be made for the establishment of ____ test locations within the treatment area. *(Frequency of testing is typically one test location per 225 square meters.)*

3. Minimum "N" values from SPT work, or cone resistance values for CPT work, should be attained in accordance with specified criteria for acceptance of the vibro-compaction work. Should these minimum values not be met, the vibro contractor should place additional probes to attain the soil improvement criteria at no additional cost to the owner.

F. SUBMITTALS

1. The Contractor shall furnish the following items to the engineer for review and approval 30 days prior to the commencement of the work.
    a. Completed project list verifying experience requirements.
    b. Details of the equipment and procedures to be used to achieve the specified results.

2. The Contractor should furnish a shop drawing to the engineer (for review) a minimum of 14 days prior to the work indicating the spacing, location, and depth of the vibro points to achieve the criteria outlined in this specification.

3. The Contractor should furnish the location, capacity, and size of the settling pond to be located outside the work area.

4. The Contractor should submit a daily log to the Engineer. This should include a recording of compaction point number, start/finish time of compaction point, depth of treatment, approximate backfill quantities per compaction point, duration of penetration, and extraction and vibratory power consumption.

5. The Contractor should submit a data logger printout showing amperage and depth over time.

6. Any change in the predetermined vibro program necessitated by a change in the subsurface conditions should be immediately reported and submitted to the Engineers.

G. MEASUREMENT AND PAYMENT

Lump sum for _____. Payment may alternately made on the basis of cubic meters of material acceptably compacted to the pay lines indicated in the Drawings. *(Performance specifications should delineate the area and depth of densification, as well as the criteria to be achieved. On this basis, the experienced Contractor can evaluate the subsurface and project conditions to provide a lump sum price for the work. Under a performance specification, the lump sum price would typically include all required backfill and testing. Unit area or volume prices should also be provided to address changes in plan quantities.)*

Mobilization/demobilization are typically paid as separate lump sum.

5.3 QUALITY CONTROL AND INSPECTION

The quality control 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 (0.5 – 1.0 m) depends on the soil reaction and is shown on the right side of Figure 12, 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 12.

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.

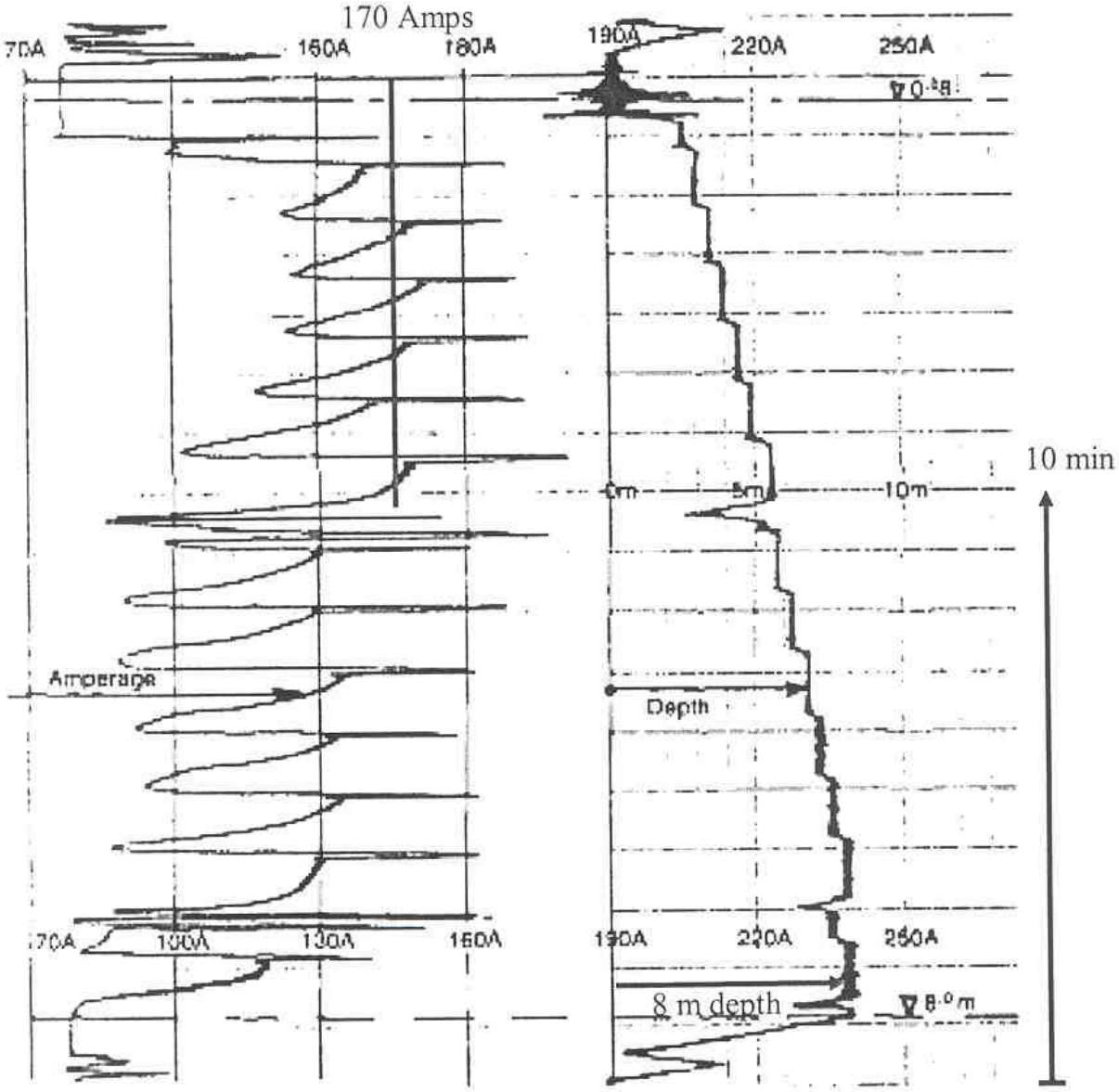


Figure 12. Typical Data Logger Results, (a) Amps versus Depth and (b) Time versus Depth.[24]

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 1.5 m (5 ft.) 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.

CHAPTER 6
COST DATA

This chapter discusses typical costs and concludes with a summary of selected vibro-compaction projects for highway applications.

6.1 BUDGET ESTIMATE

Using the criteria described in Chapter 4, 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 meter of vibro-compaction would be $15 when no backfill is placed around the probe and $25 when granular backfill is added. The specific backfill cost will vary significantly on a local basis. In addition, a mobilization/demobilization cost of $15,000 per rig should be added. Other costs that should be considered include the following:

- **Surface densification.** With the lack of overburden restraint, the upper meter 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.

Costs

For typical area densification problems, the cost range for a vibro-compaction solution will vary from $1.00 to $4.00 per cubic meter ($0.75 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.

Table 4 is a listing of the many factors that can affect the pricing of a vibro-compaction project.

Table 4. Factors affecting price of vibro-compaction projects.

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 meter 2) Cubic meter

The following procedure could be used for estimating the cost of vibro-compaction:

1. *Determine the performance requirement.* Chapter 4 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.* Price includes supervision, labor, equipment, tools, utilities and backfill added during compaction. (About 0.8 cubic meter of backfill added for each 1.5 linear meters of compaction.) Rate of production = 100 linear meters per vibrator per 8 hour day. Add the cost of additional fill to raise the grade.

6.2 SUMMARY OF TYPICAL PROJECTS

Table 5 is a limited summary of transportation related vibro-compaction projects where approximate unit costs are available.

Table 5. Summary of typical projects.

Project	Date	Remarks	Cost per m^3 or lm
Palm Beach Harbor Port Everglades, FL	1990	Vibro-compaction to compact up to 16.8 m (55 ft.) deep backfill behind a bulkhead wall measuring approximately 306,000 m^3.	\$1.40/m^3 (\$1.10/yd^3)
Manchester Airport Manchester, NH	1992	Vibro-compaction of 7.6-12.1 m (25-40 ft.) deep, loose, silty sand deposit in preparation for construction of new airport terminal building.	\$26/lm (\$8/lft)
Bridge Foundations South Roxna, IL	1994	For Illinois DOT, 130 vibro-compaction points under 2 bridge piers to 10.7 m (35 ft.) depth for liquefaction risk mitigation.	\$10/lm (\$3/lft)

CHAPTER 7
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.

7.1 I-90 Mt. Baker Ridge, Seattle, WA[19]

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 790-m-long roadway section abutting the Mt. Baker Tunnel, the 91.5-m-long by 9.0-m-wide footing was to be placed directly on soils previously placed for the existing highway embankment. Originally, the footing design assumed a 285 kPa (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 6 m (20 ft.) depth) could not support the 285 kPa (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 4.5-6.0 m (15-20 ft.), as shown in Figure 13. 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 3.0-6.0 m (10-20 ft.) perimeter around the entire footing. The compaction points were spaced on a 1.8 m (6 ft.) grid pattern with the design intent to limit settlement to a specified requirement of 19.1 mm (¾ in.).

Two plate-load tests were performed at selected compaction point locations during the work. A 1.8-m by 1.8-m (6 ft by 6 ft.) plate was placed directly over each of two points and loaded in increments to 930 kN (210,000 lbf.). The total load represented a uniform 285 kN/m^2 (5,960 psf) pressure on the test plate. The test work indicated that average total settlement under the working design load was approximately 12.5 mm (0.5 in.), with permanent plastic deformation upon unloading indicated to be approximately 6.5 mm (¼ in.).

Figure 13. Vibro-Compaction on Mt. Baker Ridge's Interstate 90.

7.2 Wando Terminal, Charleston, SC[8]

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 225,000 m^2 (267,000 yd^2) concrete-paved container storage yard. The site of the expansion section was located north of the existing facility (Figure 14). The storage yard for the expansion was divided into three areas: Area A (44,500 m^2 (53,000 yd2)), Area B (72,900 m^2 (87,000 yd^2)), and Area C (109,300 m^2 (131,000 yd^2)). Areas A and B were formerly marshlands that, over 10 years ago, had been filled to elevation +6.7 m (22.0 ft.) 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 50-100 mm (2-4 in.) 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 15. 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 16, 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.

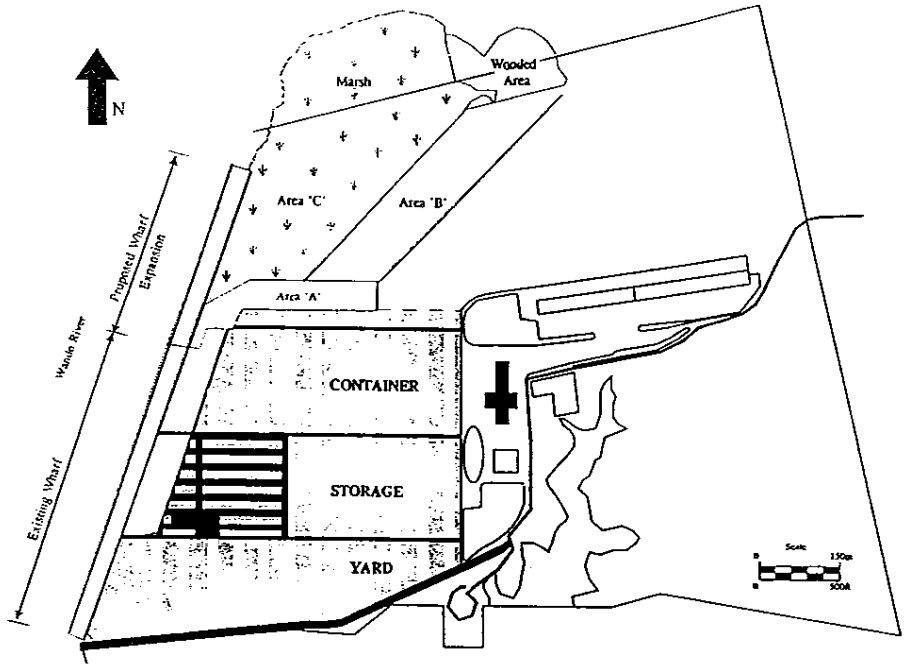


Figure 14. State Pier 41, Wando Terminal.

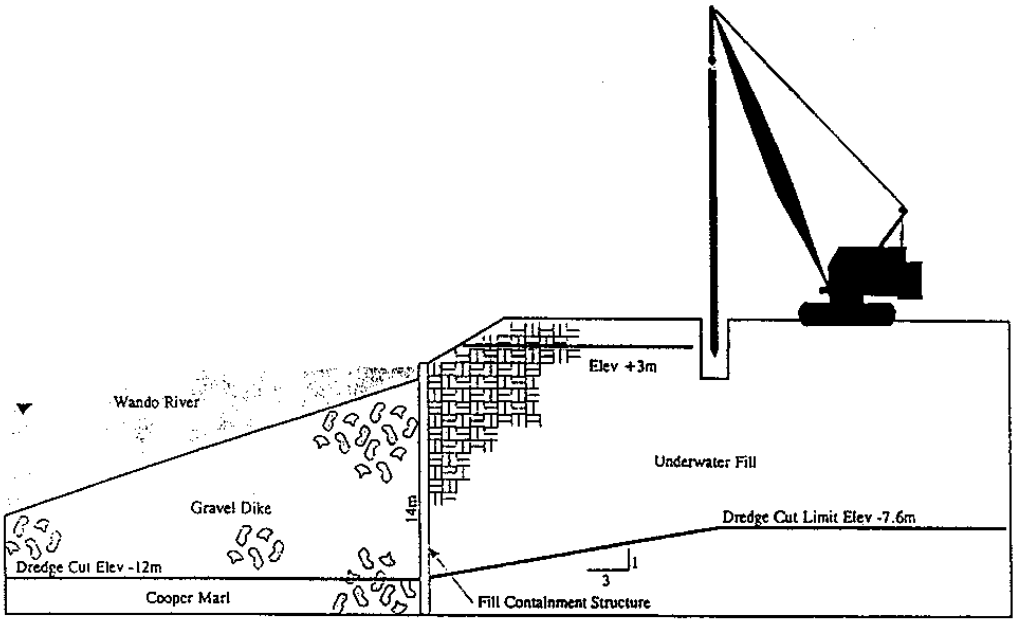


Figure 15. Vibro-Compaction at Wando Terminal.

It was determined that to achieve acceptable results, the required foundation improvement program would have to be a three-step process:

1. Dredging the soft clay to elevation -7.5 m (-25 ft.) and replacing that material with 1.2 million cubic meters of clean sand to elevation +3 m (+10 ft.).
2. Installing vertical drains (wick drains) to accelerate the consolidation of the underlying clays.
3. Transforming the very loose sand backfill into dense sand using vibro-compaction (Figure 16).

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 -7.5 m (-25 ft.) 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 -12.2 m (40 ft.) MLW on 1.5 m (5 ft.) centers, it was estimated that the maximum post-construction settlement of the container yard would include 90-180 mm (3.5-7 in.) of primary consolidation over the first 2 years, and up to 100 mm (4 in.) of secondary compression over the next 50 years.

Once the specifics of the program were determined, the backfilling of the 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 17). At the completion of this process, the sand backfill was completely densified, lowering the surface elevation from 10.7 m to 9.5 m (35 ft. to 31 ft.).

Pre-treatment and post-treatment cone penetrometer test results (Figure 18) confirmed that the required minimum densification had been achieved. Vibro-compaction densification stabilized the soils to support the weight of the containers. 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).

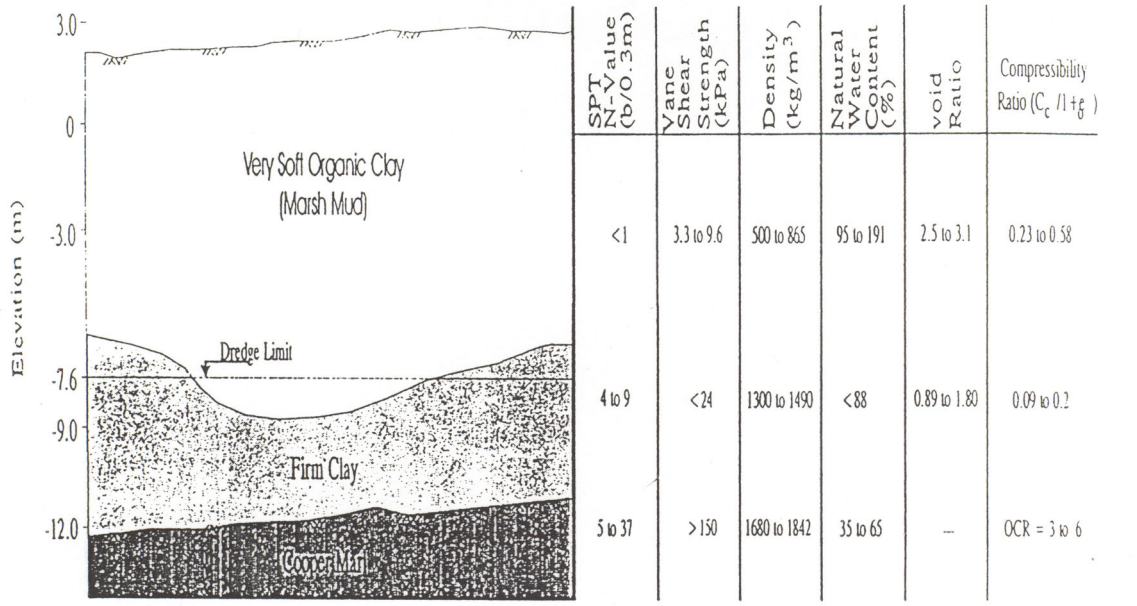


Figure 16. Generalized Subsurface Profile of Area C.

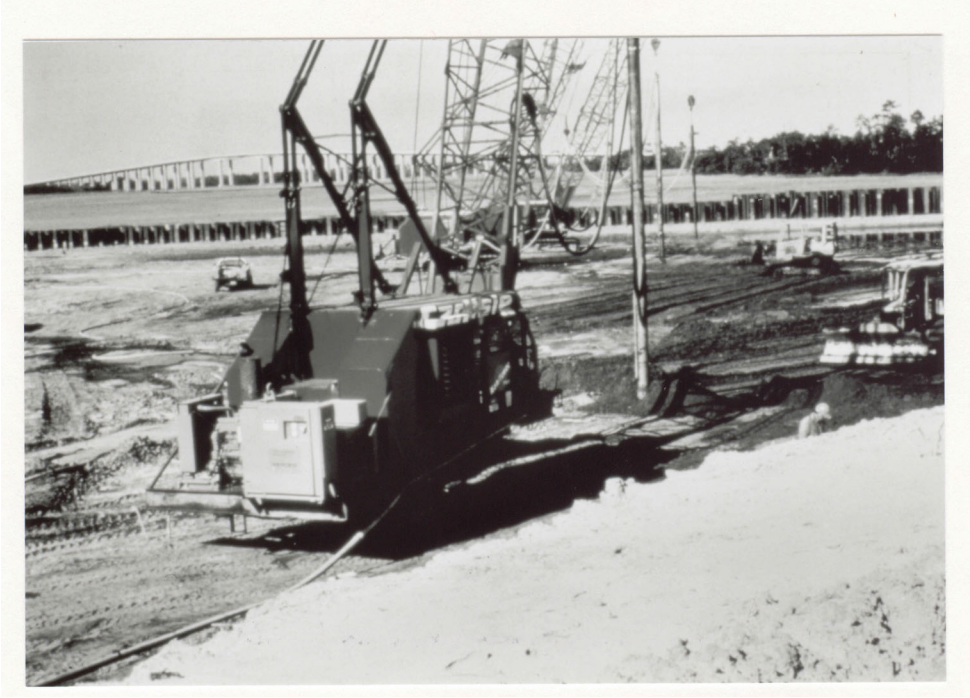


Figure 17. Densification of Loose Sand Backfill using Vibro-Compaction at Wando Terminal.

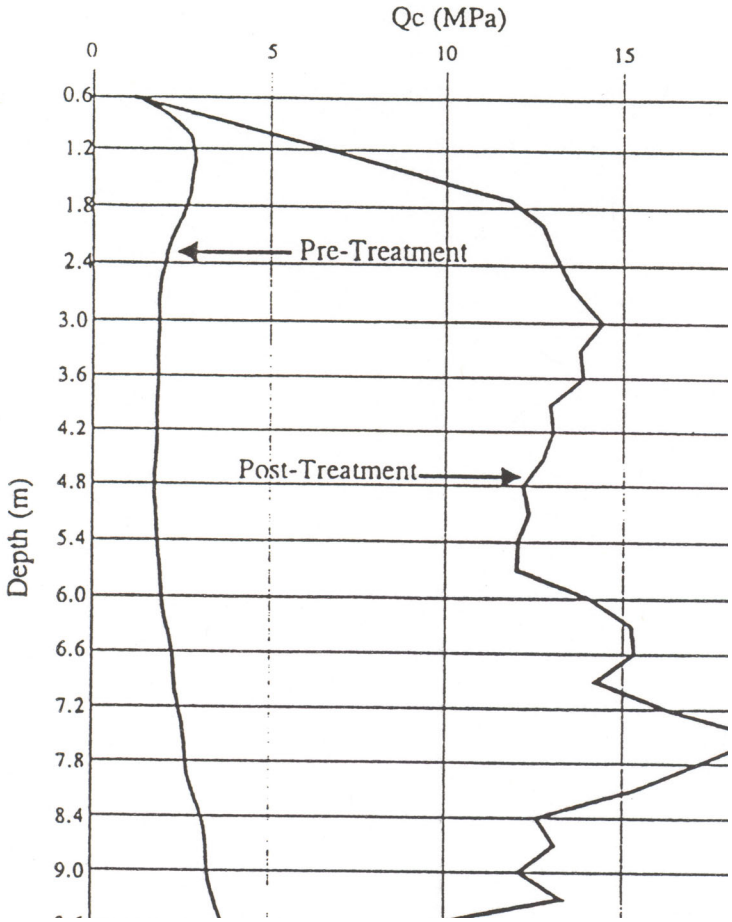


Figure 18. Sample Cone Penetrometer Test Results.

The uniqueness of this large, multi-step project was twofold. First, its size (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.

7.3 Manchester Airport, New Hampshire[20]

Construction of a new, 15,000-square-meter (160,000 ft^2) 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 3.7-13.7 meters (12 – 45 ft.). 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.[21]
2. Correlation of SPT values to volumetric strain.[22]

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 0.3 m (1 ft.) 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 12 mm (0.5 in.), with a 25-mm (1 in.) allowable total settlement.

To meet these criteria, compaction points were located on a 3 m by 3 m (10 ft. by 10 ft.) grid. The necessary depths of compaction were determined to be 8 meters and 11 meters. Although design borings had identified potentially liquefiable soils to 13.7 m (45 ft.), 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 900-square-meter 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 3.7 m to 5.2 m (12 ft. to 17 ft.), 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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Technical Summary # 4:

DYNAMIC COMPACTION

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CHAPTER 1
DESCRIPTION AND HISTORY

1.1 DESCRIPTION

Dynamic compaction is a method of ground improvement 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 5-18 Mg (11 – 40 kip) at heights ranging from 9-30 m (30-100 ft.). 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 about 3-11 m (10-36 ft.) 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.

Figure 1 is a schematic drawing that illustrates the dynamic process. The arrows represent energy that is transmitted into the soil mass following impact of the tamper. The decaying sinusoidal line at ground surface represents vibrations transmitted to adjacent areas. The predicted depth of improvement is shown as a function of the energy of a single drop.

If ground improvement is needed to provide a suitable bearing stratum for an embankment or a structure, dynamic compaction may be a viable solution.

- 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.

- 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.

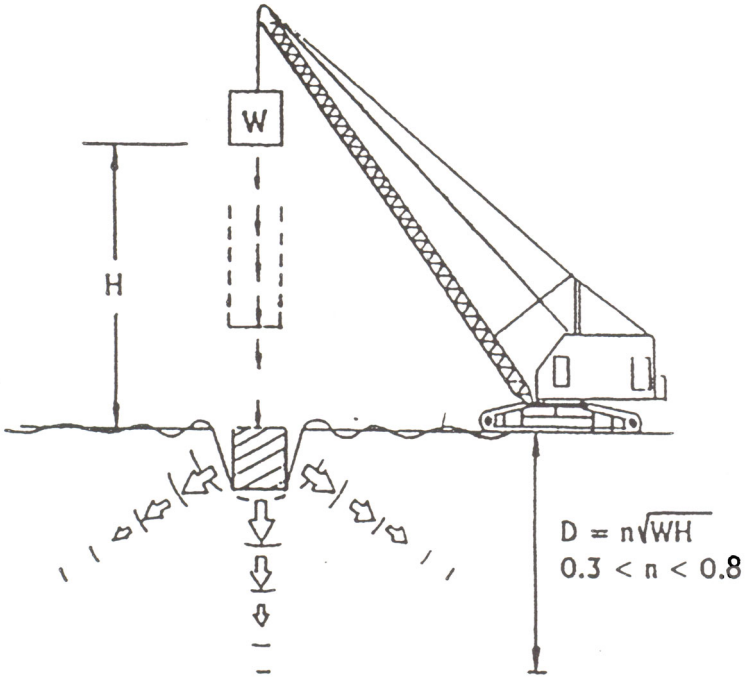

Figure 1. Schematic Illustration of Dynamic Compaction.

1.2 HISTORICAL OVERVIEW

Impacting soil deposits with tampers dropped from varying heights has been used for centuries. The Romans reportedly used this process to densify loose soils.(1) The first report on this process was published in 1812, when Rondelet published in a German book of 5 volumes titled *Kunst Zu Baun*.(1) Cohesionless soils in Germany were compacted with a 1.8-Mg (4 kip) tamper and a 1.5-m (5-ft) drop from a steam shovel in 1933.(2) The Corps of Engineers experimented with heavy tamping at the Franklin Falls Dam construction site in 1936.(3) In 1955, dynamic compaction was used in South Africa to densify loose soils to support a 76 m (250 ft.) diameter crude oil tank.(4) In Russia, heavy tampers were used to compact loessial silty and sandy soils, beginning about 1960.(5)

Use of dynamic compaction began on a regular and continuing basis in Europe in 1969 and in the United States in 1971. In Europe, tampers of 8-10 Mg (18-22 kips) were dropped from heights of 8-12 m (26-39 ft.) 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.(6)

In the United States, densification was initially achieved using tampers in the range of 1.8-5.4 Mg (4-12 kips) with drops of 6.1-10.7 m (20-35 ft.) to densify loose rubble fill and

granular deposits to support lightly loaded structures. Later, tampers up to 15 Mg (33 kip) were used to densify former landfills. This technique was initially called "pounding," but eventually became know as dynamic compaction.[7]

Before 1975, European and American practices developed independently and were somewhat experimental. In 1975, a technical article was published that dealt with the theoretical and practical aspects of dynamic compaction.[6] 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 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 dynamic compaction. In the United States, the state of practice was summarized by Lukas in FHWA/RD-86/133, "Dynamic Compaction for Highway Construction" which presented guidelines for implementation.[7] Guidelines for implementation have been kept current in FHWA publications released in 1995[8], 1998[9], and this 2004 manual.

1.3 FOCUS AND SCOPE

The purpose of this technical summary is to provide an overview of this process and to acquaint the reader with dynamic compaction. The subjects discussed include

- applications and limitations
- preliminary design and construction concepts
- construction monitoring
- specifications
- representative case histories

FHWA-SA-95-067, Dynamic Compaction, provides a comprehensive discussion regarding all of the factors that should be considered for planning and implementing a dynamic compaction project.[8]

1.4 GLOSSARY

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. The typical units are tonne meters per meter squared (tm/m^2), or KiloJoules per meter squared (kJ/m^2).

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.

CHAPTER 2
DESIGN CONSIDERATIONS

2.1 INTRODUCTION

A number of fundamental questions must be addressed before proceeding with an in-depth evaluation of 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?

2.2 APPLICATIONS

The primary purpose of dynamic compaction is to densify natural and fill deposits to improve soil properties that affect performance. The primary uses of dynamic compaction to date include

- densification of loose deposits
- collapse of large voids
- related applications

a. Densification of Loose Deposits

The primary use of dynamic compaction is to densify loose deposits so as to reduce the settlement under load application. Densification results from the soil grain particles being compacted to reduce the 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 restructure into a denser state of packing at a lower water content.

In areas subject to seismic activity, sands or silty sand deposits that are stable under static conditions could liquefy during a seismic event. Dynamic compaction has been used to densify these deposits, which makes them non-susceptible or less-susceptible to liquefaction from earthquakes.

In arid regions, partially saturated collapsible soil deposits may be present. These soils have been deposited naturally, with a large void ratio that is temporarily held in this loose state of packing by weak cementation. Upon wetting, the cementatious bonds between the particles are dissolved and the soils collapse into a denser state of packing. This results in settlement of roadways constructed over these deposits. Dynamic compaction can be used to overcome the cementatious bonds and move the particles into a more dense condition to reduce or eliminate settlement upon wetting.

Figure 2 illustrates the soil behavior during 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 2, 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.

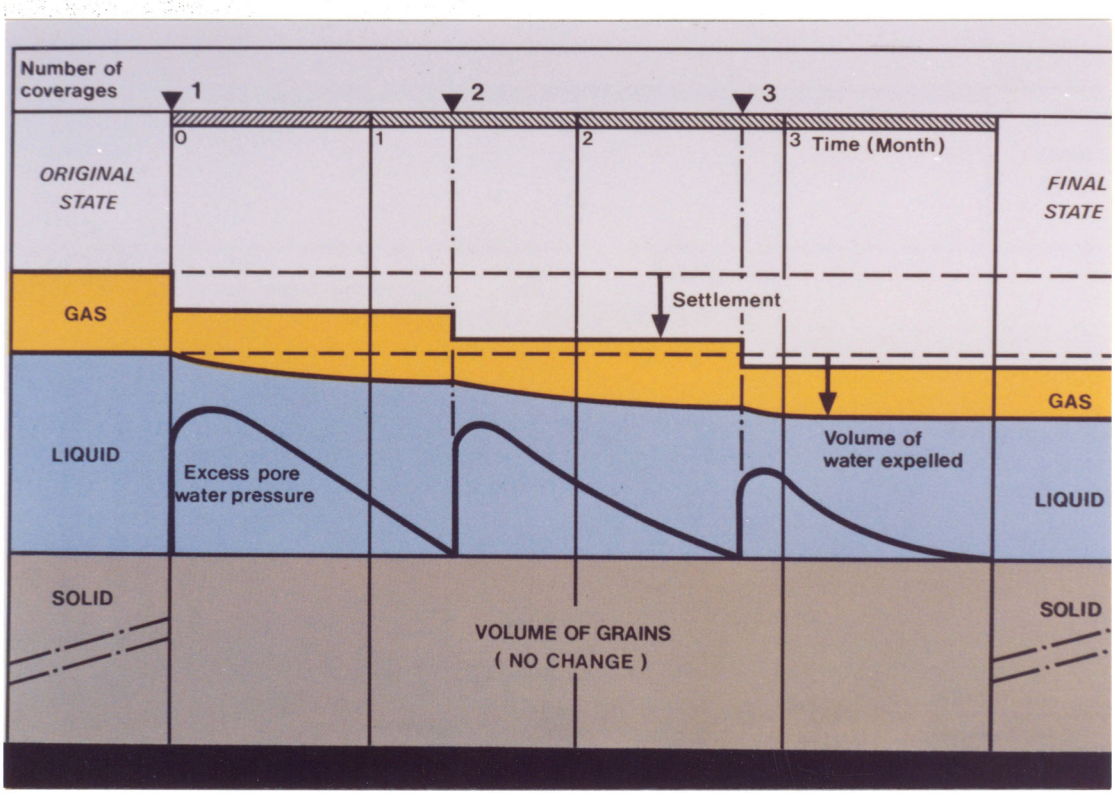

Figure 2. Phase Diagram Model of Soil Behavior during Dynamic Compaction
(Courtesy of Soletanche.)

The final state of the soil mass is shown on the right side of Figure 2. 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.

b. Collapse of Large Voids

Large voids may exist within the soil mass in either natural or fill deposits. 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 dynamic compaction. For dynamic compaction to be effective, the void should be located within the depth of treatment as determined by Equation 1 of Chapter 4.

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 dynamic compaction, 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.

At sites where there are known or suspected large voids, provision should be made for application of additional energy on a localized basis. Excessive crater depths or larger than normal ground depressions indicate the presence of these voids. Close field monitoring is required at these sites.

c. Related Applications

In weak saturated soils relatively deep craters (> 1.5 m (5 ft.)) 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 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 3 to 4 meters (10 to 13 ft.).

2.3 ADVANTAGES AND DISADVANTAGES

The advantages of 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 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 5-10 percent of the thickness corresponding to the depth of treatment, (as predicted by Equation 1 of Chapter 4), occurs during densification. In extremely loose fills, such as recent landfills, the ground compression can be 20-25 percent.

- 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. 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 nearly 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 dynamic compaction on a local basis, making the cost for 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.

- 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 dynamic compaction.

Disadvantages of dynamic compaction include the following:

- 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 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 2 m (6.6 ft.) below ground surface. If the water table is higher than 2 m (6.6 ft.), 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 dynamic compaction operation.

- Lateral ground displacements of 25 to 76 mm (1 – 3 in.) have been measured at distances of about 6 m (20 ft.) from the point of impact of 15- to 30-Mg (33-66 kip) tampers. Utilities or buried vessels within the zone of influence could be displaced or damaged.

2.4 SUITABILITY OF DYNAMIC COMPACTION FOR VARIOUS DEPOSITS

The suitability of 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

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 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 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 dynamic compaction require a lengthy period of time to dissipate, which renders 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 2.2.c).

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. 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.

Figure 3 is a guide for estimating the suitability of dynamic compaction for various soil deposits, based upon conventional index tests. Three categories of deposits shown on this figure are summarized below.

- Zone 1 represents pervious soil deposits where 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.

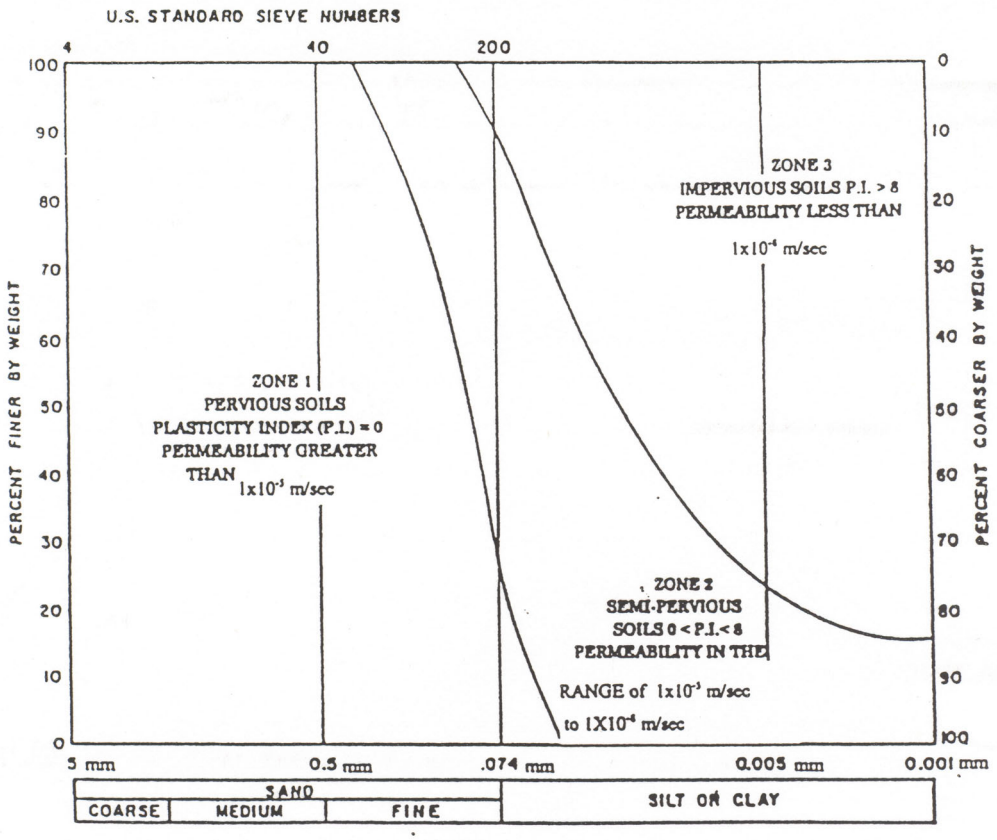

Figure 3. Grouping of Soils for Dynamic Compaction.

- Zone 3 represents impervious clayey deposits. The permeability of these soils is generally less than 10^{-8} m/s. 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 dynamic compaction is to first determine the grain size distribution of representative samples to determine where the gradation plots on Figure 3. If the gradation plots in Zone 1, proceed with the design and construction sequence for 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, dynamic compaction will work, but the energy should be applied in phases.

On some projects, test sections are used to evaluate the effectiveness of 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 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.

2.5 LIMITATIONS AND ENVIRONMENTAL CONSIDERATIONS

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 dynamic compaction is undertaken close to property lines, ground vibrations transmitted off site should be considered.

The U.S. Bureau of Mines has found that building damage is related to particle velocity.[10] Figure 4 was developed by the Bureau based on experiences with damage measurements made in residential construction from blast-induced vibrations. The limiting particle velocity depends upon the frequency of the wave form. Normally, dynamic compaction results in frequencies of 5-12 Hz. Using Figure 4 as a guide, this would limit peak particle velocities to values of 13 mm/s (½ in./s) for older residences with plaster walls and 19 mm/s (¾ in./s) for more modern constructions with drywall. Peak particle velocities that exceed the values given in Figure 4 do not mean that damage will occur. Rather, these values are the lower threshold beyond which cracking of the plaster or drywall may occur.

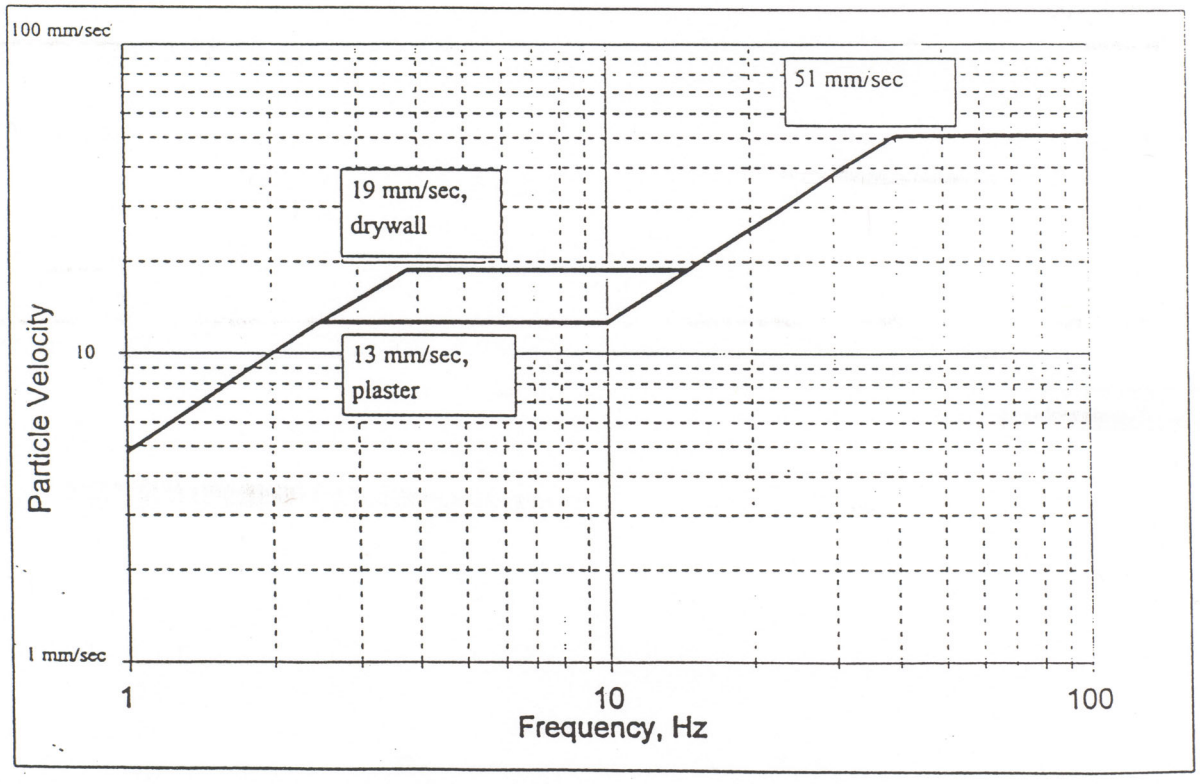

Figure 4. U.S. Bureau of Mines Safe Level of Blasting Vibrations for Houses.

Data generated by the U.S. Bureau of Mines indicate that minor damage occurs when the particle velocity exceeds 51 mm/s (2 in./s), and major damage occurs when the particle velocity exceeds about 193 mm/s (7.6 in./s). Thus, keeping the particle velocity less than about 13-19 mm/s (½ to ¾ in./s) 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 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 5 can be used. 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. Figure 5 has been prepared on the basis of actual readings obtained at specific sites.

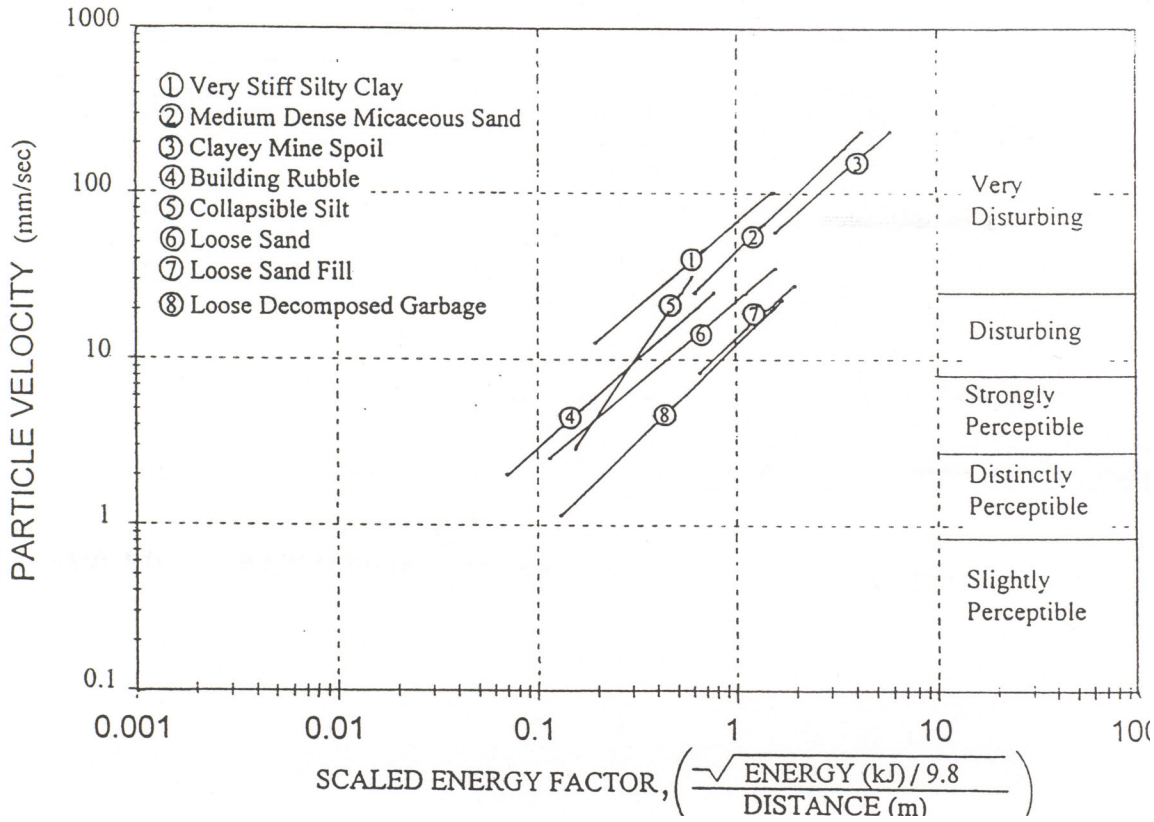

Figure 5. Scaled Energy Factor Versus Particle Velocity.[7]

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 5. Ground vibrations would be disturbing to people in the range of 13-19 mm/s (½ to ¾ in./s). 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 76 mm/s (3 in/s) without damage.

Dynamic compaction also causes lateral ground shifting. Figures 6 and 7 indicate measurements taken for three different soil types, with inclinometers located at 3 m and 6 m (10 ft. to 20 ft.) from the point of impact of tampers in the range of 15-29 Mg (33-64 kips). At a distance of 3.0 m (10 ft.) from the point of impact, lateral deformations as much as 254 to 305 mm (10-12 in.) were observed, while at 6 m (20 ft.), lateral deformations ranging from 13-76 mm (½-3 in.) 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 dynamic compaction operations with tampers ranging from 15-30 Mg (33-66 kips) should not be undertaken within about 6-7 m (23-26 ft.) 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.

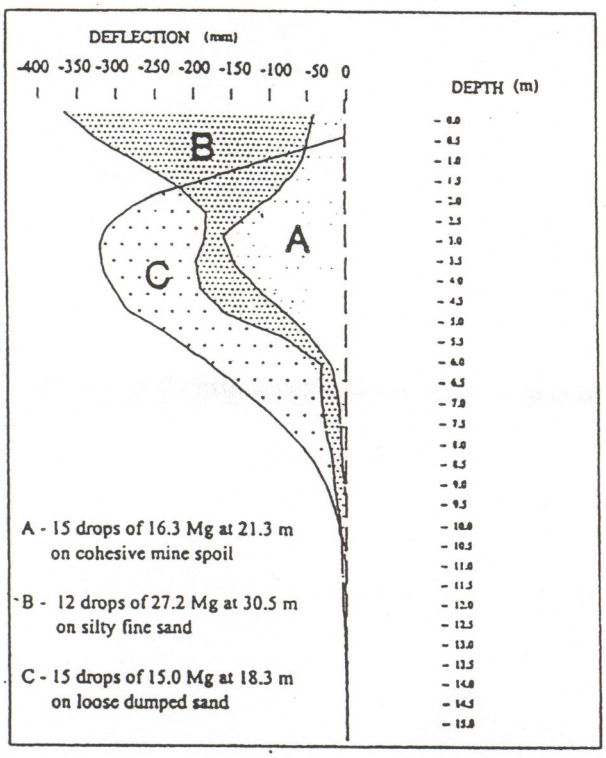

Figure 6.
Lateral Movements at 3 m (10 ft.) Drop Point.

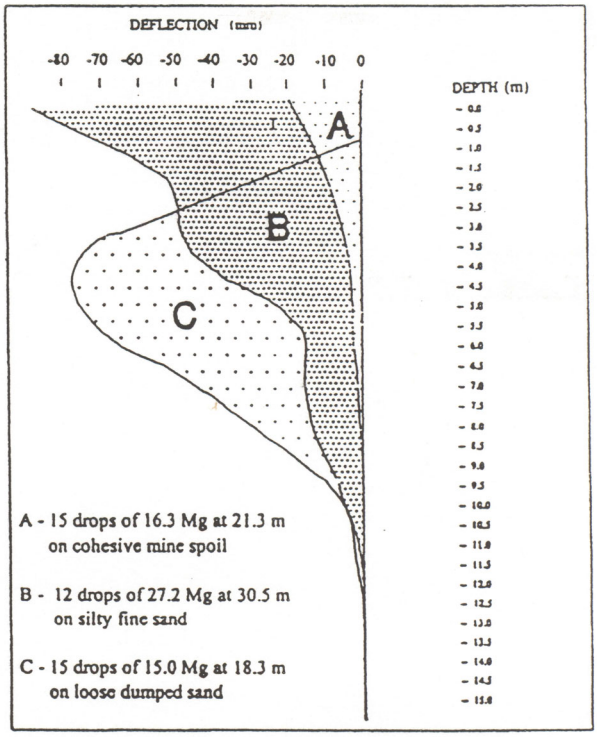

Figure 7.
Lateral Movements at 6 m (20 ft.) from From Drop Point.

Where 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.(7)

There is a depth limitation on improvement. The heaviest tamper that can be lifted with conventional equipment is about 18 Mg (40 kips) with a drop height of 23-27 m (75-90 ft.). This will result in a maximum improvement depth of about 11 m (36 ft.). If deeper improvement is necessary, dynamic compaction in combination with other systems, such as grouting and stone columns, should be considered.

2.6 ALTERNATE GROUND IMPROVEMENT METHODS

Before deciding upon ground improvement by dynamic compaction, alternate foundation solutions should be investigated in order 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.

- *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.

- *Vibroreplacement could be considered* to stiffen clayey soil deposits and vibrocompaction to densify granular soils.

- *Deep soil stabilization techniques,* such as jet grouting, deep soil mixing, or compaction grouting, can be considered.

CHAPTER 3
CONSTRUCTION AND MATERIALS

3.1 INTRODUCTION

Conventional lifting cranes are used for dynamic compaction projects where the tamper size is less than about 18 Mg (40 kips). Tampers are sometimes built especially for 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 dynamic compaction projects, many general, demolition, or excavation contractors will perform dynamic compaction when properly guided and monitored.

3.2 TAMPERS

Dynamic compaction generally is performed with tampers ranging from 5-27 Mg (11-60 kips). The lighter tampers are used where the thickness of the deposit is relatively thin, such as 3 m (10 ft.), and the heaviest tampers are used where the deposit is about 9-12 m (30-40 ft.) 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 36-72 kPa (5.2 to 10.4 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 36 kPa (5.2 psi) will distribute the energy over too wide an area to cause deep densification.

Figures 8 to 11 are photos of 5.5, 13.6, 14.0, and 32 Mg (12.1, 30.0, 30.9, and 70.6 kips) tampers, respectively. The 5.5-Mg (12.1 kips) tamper has a round 152 mm (6 in) thick steel base plate and a 25.4 mm (1 in) thick steel cylinder that is filled with concrete. The 13.6-Mg (30 kips) 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 32.0-Mg (70.6 kips) 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 10-34 kPa (1.5-4.9 psi). Figure 12 shows a tamper used for the ironing pass. On some projects, surface densification is achieved with either conventional compaction equipment or by proofrolling with a fully loaded dump truck.

3.3 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. Figures 13 to 15 show typical equipment used to raise and drop the 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 5-18 Mg (11-40 kips). Table 1 shows the type of crane and cable size required to repeatedly raise and drop tampers of different sizes. For tamper weights in the range of 18-23 Mg (35-51 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.

Figure 8. 5.5 Mg (12.1 kips) Tamper.

Figure 9. 13.6 Mg (30.0 kips) tamper.

Figure 10. 14.0 Mg (30.9 kips) Tamper.

Figure 11. 32.0 Mg (70.6 kips) Tamper.

Figure 12. Tamper with Low Contact Pressure for Ironing Pass.

Figure 13. Lifting Crane of 36 Mg (70 kips) Rated Capacity Used with 5.5 Mg (12 kips) Tamper.

Figure 14. Dragline-Type Lifting Crane with 32 mm (1¼ in) Cable for use with 18.2 Mg (40 kips) Tamper.

Figure 15. Manitowoc 4000 Crane with Special Lifting Drum Attached to Rear of Crane for Use with 29 Mg (64 kips) Tamper.

Table 1. Equipment requirements for different sized tampers.

Tamper Weight	Crawler Crane Size	Cable Size
3.5-7.5 Mg (3.8-8.3 tons)	36.3-45.7 Mg (40-50 tons)	19-22 mm (¾-⅞ in)
7.5-13 Mg (8.3-14.3 tons)	45.7-90.7 Mg (50-100 tons)	22-25 mm (⅞-1 in)
13.5-16.5 Mg (14.9-18.2 tons)	90.7-113 Mg (100-125 tons)	25-29 mm (1-1⅛ in)
16.5-23 Mg (18.2-25.3 tons)	136-159 Mg (150-175 tons)	32-38 mm (1¼-1½ in)

Specialized lifting equipment has been devised for tampers 23-27 Mg (50-60 kips). Figure 16 shows a crane developed for large-sized tampers. The lifting drum is approximately 1.5 m (5 ft.) 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.

Figure 17 shows a crane using multiple part lines to lift a 29 Mg (64 kips) tamper that is then allowed to free fall and impact the ground.

Figure 16. Lifting Crane Built Only for Dynamic Compaction with 1.5 m (4.9 ft.) Diameter Lifting Hoist and Single Line-Rated Capacity of 45 Mg (50 ton) at Line Speeds of 26 m/min (85 ft/min) Used with 29 Mg (32 ton) Tampers.

Figure 17. Multiple Part Line Used to Lift 29 Mg (64 kips) Tamper *(below grade in photo)*.

3.4 SURFACE STABILIZING LAYER

Where soft ground conditions prevail, it may be necessary to place a surface stabilizing layer to allow travel of the 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 152 mm (6 in.) maximum size, down to sand size. The thickness of these layers depends upon the stability of the surface deposits, but thicknesses ranging from 0.3-0.9 m (1-3 ft.) have been used successfully. Figure 18 shows dynamic compaction equipment working on a site where crushed rock was placed to a depth of 0.9 m (3 ft.) over 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 dynamic compaction, it is used only where absolutely necessary.

3.5 CONSTRUCTION SEQUENCING

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 7.6 m (25 ft.) 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.

Figure 18. Dynamic Compaction Over a 0.9 m (3 ft.) Layer of Crushed Rock Placed on Landfill.

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.

CHAPTER 4
DESIGN CONCEPTS

4.1 PROBLEM DEFINITION

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

- determining the project performance requirements for the completed structure,
- selecting the tamper mass and drop height to correspond to the required depth of improvement,
- estimating the degree of improvement that will result from dynamic compaction,
- determining 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. FHWA-SA-95-037 provides additional details.

4.2 PERFORMANCE REQUIREMENTS

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 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 150 mm (6 in) 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.[11] 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 150 mm (6 in). If so, 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 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, dynamic compaction could reduce the risk of differential settlement from these unforeseen pockets of loose ground. Since 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.

4.3 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:

$$D = n\,(WH)^{1/2} \qquad (1)$$

Where: D = depth of improvement in meters

n = empirical coefficient that is approximately 0.5 for most soils. As shown in Figure 19, n varies between 0.3 and 0.8. 0.4 is typically used for landfills

W = mass of tamper in Megagrams (metric tonnes)

H = drop height in meters

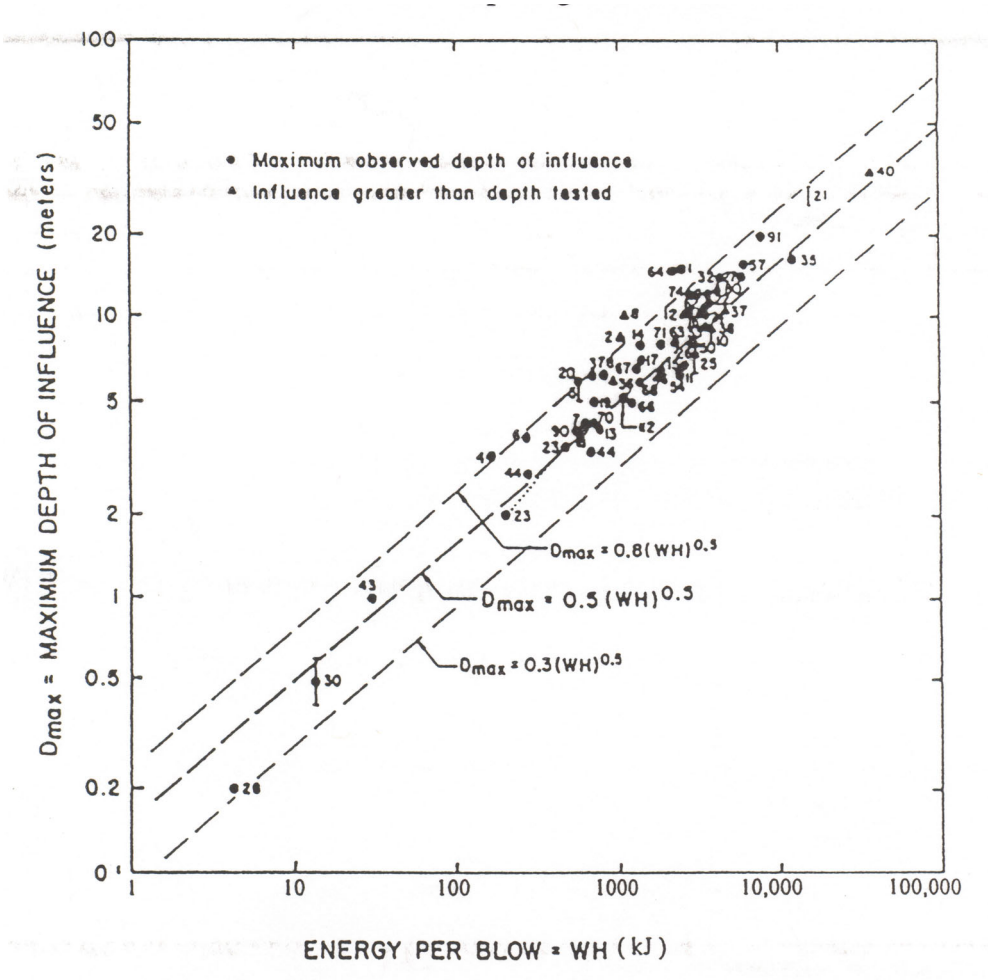

Figure 19. Trend between Apparent Maximum Depth of Influence and Energy per Drop.

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 Reference (7).

Various combinations of W and H can be used in Equation 1, depending upon equipment availability. Figure 20 summarizes the relationship between drop height and tamper mass that has been used on numerous projects. Figure 20 can be used as a starting point with adjustments in W and H made after a contractor is selected.

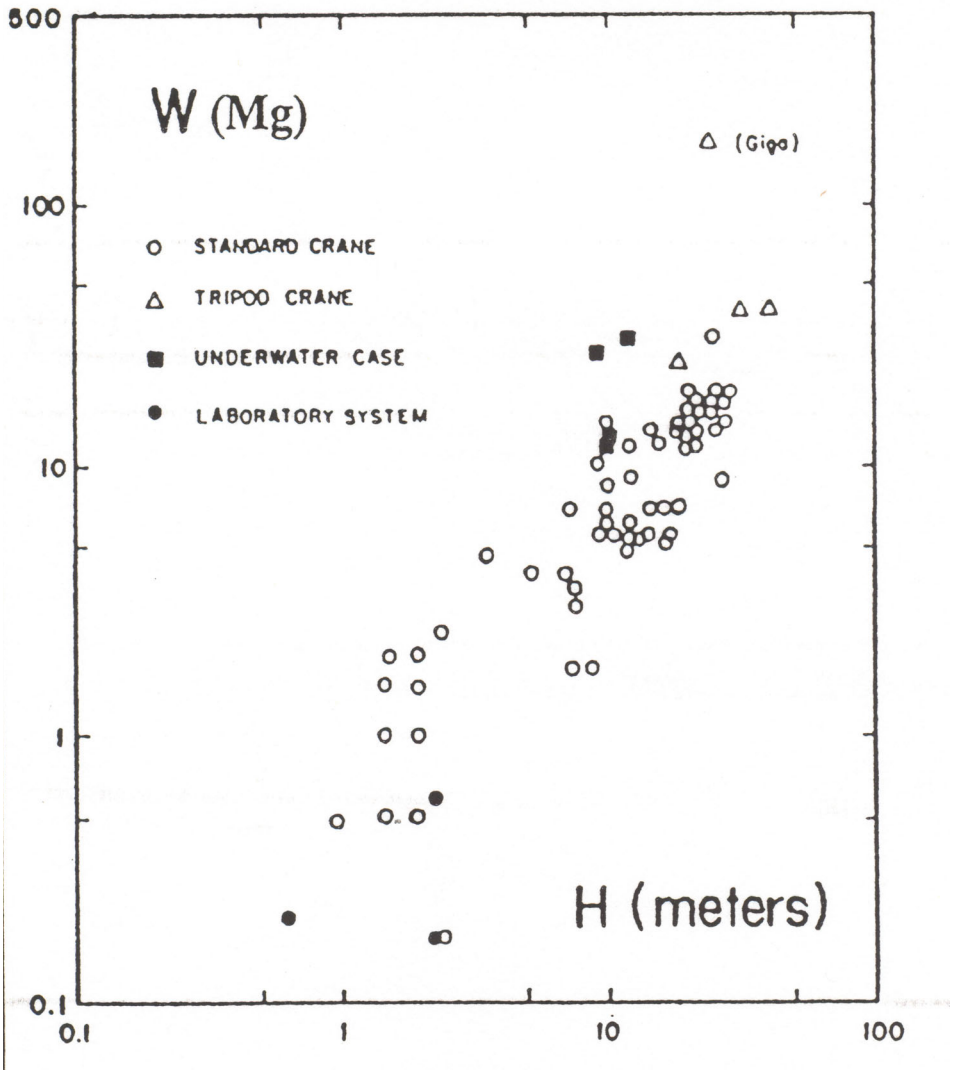

Figure 20. Relationship between Tamper Mass and Drop Height.[12]

The amount of improvement resulting from dynamic compaction is generally measured by conventional in-situ testing techniques such as SPT, CPT, or PMT. Test values obtained after dynamic compaction are compared with initial values before 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. Figure 21 illustrates the variation of improvement with depth.

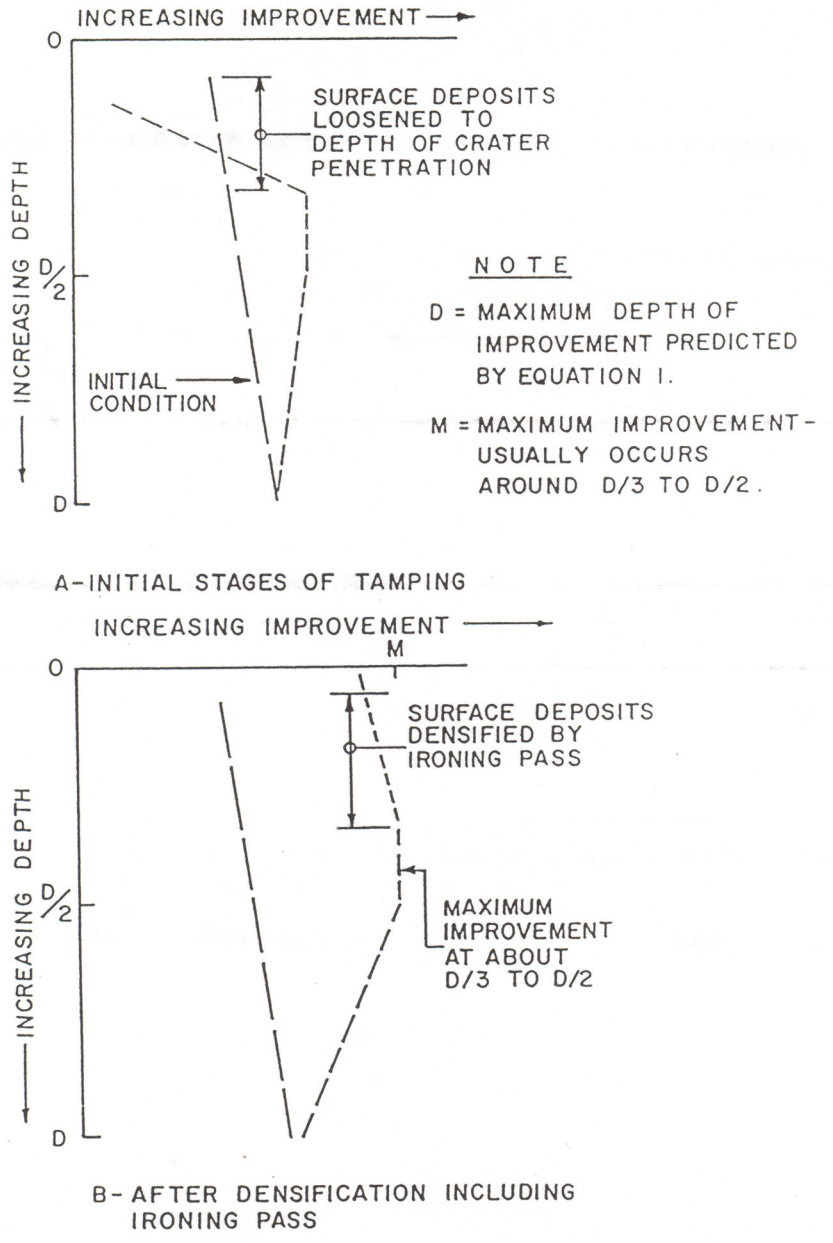

Figure 21. Variations in Improvements with Depth During Dynamic Compaction.

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 2.[7] 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 2-3 MJ/m^2 (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 2.

Table 2. Upper bound test values after dynamic compaction.

	Maximum Test Values		
Soil Type	**Standard Penetration Resistance (blows/300 mm)**	**Static Cone Tip Resistance (MPa)**	**Pressuremeter Limit Pressure (MPa)**
Pervious coarse-grained soil:			
sands & gravel	30-50	19-29	1.9-2.4
Semipervious soil:			
sandy silts	25-35	13-17	1.4-1.9
silts & clayey silts	20-35	10-13	1.0-1.4
Partially saturated impervious deposits: clay fill & mine spoil	20-40*	N/A	1.4-1.9
Landfills	15-40*	N/A	0.5-1.0

** Higher test values may occur because of large particles in the soil mass.*

The upper-bound test values given in Table 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 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 dynamic compaction, after all excess pore water pressures have dissipated. Lukas[13] reported on 5 projects where improvements in soil properties were measured years after dynamic compaction. The improvements in standard penetration or pressuremeter tests ranged from 15-200 percent. Schmertman[14] states that improvements of 50-100 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 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 Reference (14).

4.4 ENERGY REQUIREMENTS

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:

$$AE = \frac{(W)(H)(N)(P)}{(grid\ spacing)^2} \tag{2}$$

Where:

AE = applied energy in tm/m^2

N = number of drops at each specific drop point location

W = tamper mass in Mg (metric tonnes)

H = drop height in m

P = number of passes

Grid spacing in m

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

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 1-3 MJ/m^2 (21,000-62,000 ft-lbf/ft^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 3 in units of volume. 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.

Table 3. Applied energy guidelines.

Type of Deposit	Unit Applied Energy kJ/m^3 (ft-lbf/ft^2)	Percent Standard Proctor Energy
Pervious coarse-grained soil (Zone 1 of Figure 3)	200 – 250 (4,100 – 5,200)	33-41
Semipervious fine-grained soils (Zone 2 and clay fills above the water table and Zone 3 of Figure 3)	250 – 350 (5,200 – 7,200)	41-60
Landfills	600 – 1100 (12,400 – 22,700)	100-180
Note: Standard Proctor energy equals 600 kJ/m^3 (12,400 ft-lbf/ft^2)		

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 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.

An example helps illustrate the use of Table 3 for planning purposes. Consider a site that consists of a loose rubble fill with a thickness of 8 m (26.2 ft.). 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 (250 kJ/m^3 (5,200 ft-lbf/ft^2)) would be used. Since the deposit is 8 m (26 ft.) thick, 2 MJ/m^2 (41,000 ft-lbf/ft^2) of average applied energy is required. Substituting this number into Equation 2, and using the tamper mass and drop height from Equation 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 2. The required mass of tamper, W, and drop height, H, is calculated from Equation 1. The grid spacing usually is selected as 1.5-2.5 times the diameter of the tamper. Equation 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 3 can be related to standard proctor energy (ASTM D698), which is equal to 600 kJ/m^3 (12,400 ft-lbf/ft^2). Pervious coarse-grained soils generally require about 33-41 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 100-180 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 dynamic compaction should be used to verify whether the desired improvement is reached.

CHAPTER 5
CONTRACTING PROCEDURES, SPECIFICATIONS, AND CONSTRUCTION MONITORING

This chapter presents a brief summary of methods that can be used in contracting for dynamic compaction, plus an overview of the monitoring that normally occurs during the field work. Complete construction specifications are detailed in FHWA-SA-95-037.

5.1 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 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 dynamic compaction that are necessary to achieve the proper improvement.

The method specification approach is more common as knowledge of 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 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 dynamic compaction. Since these contractors will do a significant amount of engineering and planning of the dynamic compaction operation, the cost for dynamic compaction using this approach is generally higher than for projects where method specifications are used.

5.2 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 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 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

5.3 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 dynamic compaction plan
- verification of the end product

5.4 QUALITY CONTROL/ASSURANCE MONITORING

Regardless of whether dynamic compaction is performed under the method or the performance approach, it is essential to monitor the dynamic compaction operations.

During 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 dynamic compaction includes these procedures:

- Where 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 dynamic compaction in an area should be measured. Settlement readings should be obtained on a grid basis over an area before and after dynamic compaction to determine the average induced subsidence. Generally, the induced subsidence ranges from 5-10 percent, except in landfills, where it could be higher.

- Field testing could be performed while the 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 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 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 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 dynamic compaction.

After the 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 30-60 days following completion of 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 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 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 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 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 dynamic compaction plan would need to be mutually agreed upon between the owner and the contractor. The borings and field tests made after dynamic compaction should also be monitored to confirm that the improvement has been achieved.

CHAPTER 6
COST DATA

The cost of 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 18 Mg (40 kips), costs are relatively predictable because conventional lifting equipment can be used to raise and drop the tamper repeatedly. Table 4 compares costs for dynamic compaction versus the size of tamper.

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.

Table 4. Dynamic compaction costs.

Size of tamper required Mg (ton)	**Unit cost Dollars/m^2 (Dollars/ft^2)**
4-7 (4.4-7.7)	8.00-11.00 (0.75-1.00)
7-15 (7.7-16.5)	11.00-19.00 (1.00-1.75)
15-18 (16.5-20)	19.00-25.00 (1.75-2.30)
18-91	Negotiated for each job
Note: Prices based on projects undertaken from 2000 to 2003	

Costs for 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 7-9 Mg (15-20 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 9-18 Mg (20-40 kips), heavier lifting equipment is required, so much of this work is done by specialty contractors.

When the tamper exceeds 16-18 Mg (35-40 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. Figure 15 shows a Manitowoc 4000 crane with a special hoist on the back for repeatedly raising and dropping a 29-Mg (64 kips) tamper. Figure 16 shows a specialized crane that has been developed for lifting tampers in the range of 29-45 Mg (64-100 kips). The costs for 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 1 in Chapter 4 is used to predict the proper energy. After selecting a drop height, the mass of the tamper is then determined. Table 4 can be used to estimate the cost. More refined methods of estimating costs are presented in FHWA Geotechnical Engineering Circular 1.[8]

In addition to the cost for 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 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 0.3-0.9 m (1-3 ft.) of a crushed rock or granular material. This would add to the cost for 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 5-10 percent of the thickness of the densified ground.

CHAPTER 7
CASE HISTORIES

Two case histories are presented to illustrate typical applications.

7.1 HIGHWAY EMBANKMENT CONSTRUCTED IN MINE SPOIL

Project Description

Interstate 65 in Jefferson County, Alabama, was extended over a mine spoil area for approximately 760 m (2,500 ft.). At either end of the mine spoil area, the final grades required approximately 4.6 m (15 ft.) of new fill, but in the center, there was a cut of 13.7 m (45 ft.). From this final grade, there would still be 15.2-36.6 m (50-120 ft.) of mine spoil below the roadway. The new roadway was to be a four-lane highway with a width of 55.5 m (182 ft.).

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 51 mm (2 in.).

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 0.3 m (1 ft.). There were some areas where the SPT values were as low as 5-10 blows per 0.3 m (1 ft.) and other areas as high as 25-35 blows per 0.3 m (1 ft.). The variation in SPT value at depth is illustrated in Figure 22.

Ground water was encountered at depths in excess of 30 m (100 ft.) below ground surface, and was not a factor on this project.

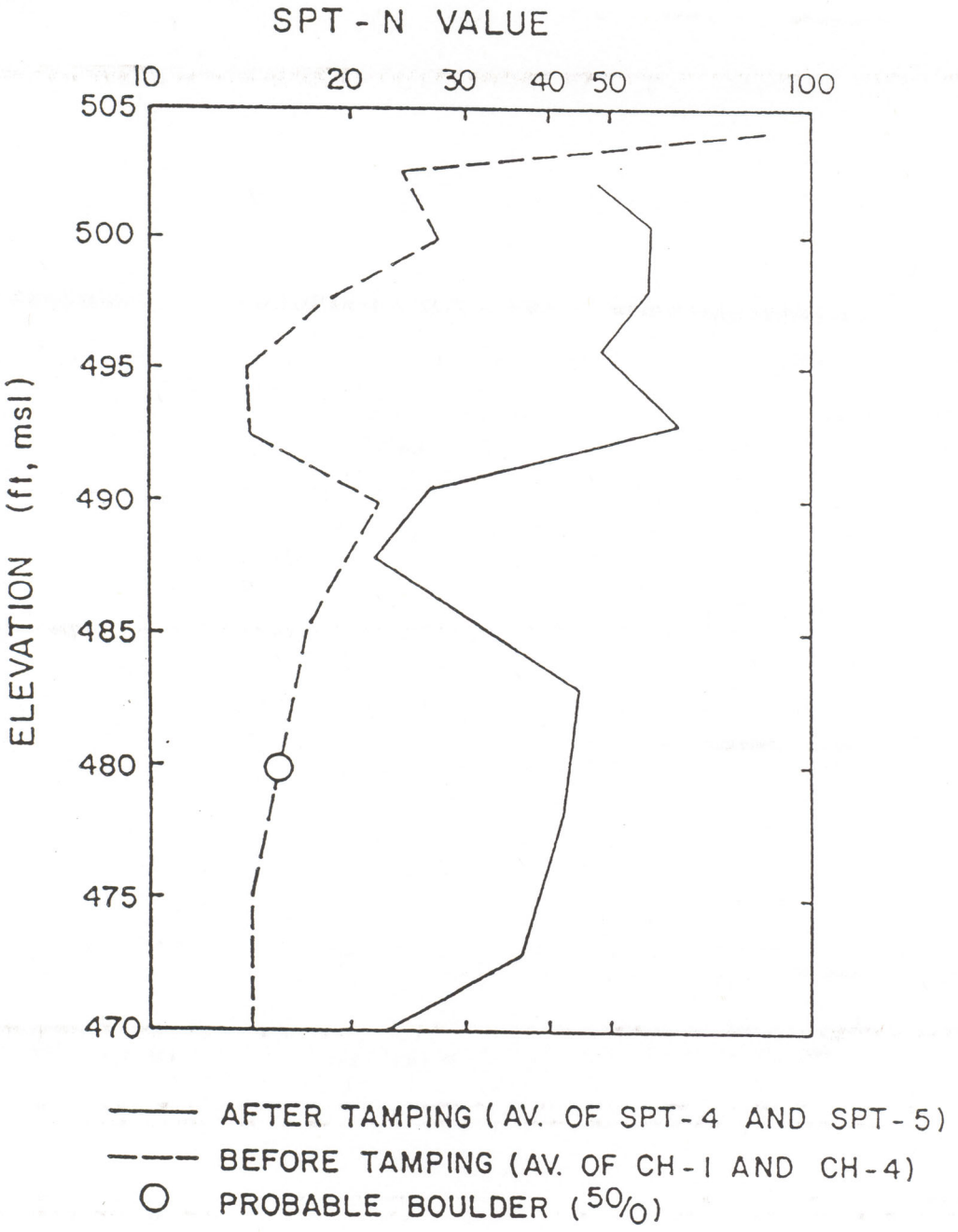

Figure 22. Increase in SPT Values in a Mine Spoil after Dynamic Compaction.

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:

1. Provide a more uniform subgrade for the new pavement.

2. Crush existing large boulders and rock fragments that are subject to deterioration.

3. Densify localized areas of loose/soft soil in the upper 7.6-9 m (25-30 ft.), which could subside after highway construction.

Excavation of 7.6 m (25 ft.) of mine spoil followed by recompaction with conventional compactors was considered an alternate to dynamic compaction. However, dynamic compaction was found to be significantly less expensive.

Predicted Densification Procedure

Using the guidelines presented in this technical summary, the first step was to calculate the tamper mass and drop height for a desired depth of improvement of 7.6-9 m.

Using D = 9 m, n = 0.5 as average and Equation 1,

$$9 = 0.5(WH)^{1/2}$$

$$WH = 324$$

From Figure 20, using W = 20 Mg, H ranges from 15-31 m; use H = 16.5 m,
since WH = 20 *x* 16.5 = 330 > 324 Mgm.

The second step was to calculate the energy to apply. Using Table 3 as a guide, for Zone 2 type deposits,

$$E = 250\text{-}350 \text{ kJ/m}^3$$

Because the deposits are already in a medium-dense condition, use E = 250 kJ/m^3. For a 9 m 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 tamper dropped 16.5 m, a reasonable spacing would be 3 m, as the tamper diameter is likely

to be 2 m. Using Equation 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.

$$(2250-250)\,kJ/m^2 = \frac{(20\,Mg)(10\,kN/Mg)(16.5\,m)(N)}{(3)^2}$$

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 2 would be 25-35 for sandy silts, and 20-35 for clayey silts. Estimated induced settlement is 5 percent (9 m) = 0.45 m.

Actual Densification Procedure

Based upon results of a test section, it was determined that densification to a depth of 7.6-9 m (25-30 ft.) could be achieved using a 20-Mg (44-kip) tamper with a drop height of 19 m (62 ft.). The high-level energy was applied using five drops at each grid point location, with a spacing of 3.0 m (10 ft.) 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 ft.), a grid spacing of 1.8 m (6 ft.), and one drop per grid point location.

This procedure resulted in an average applied unit energy of 2.1 MJ/m^2 (142 ft-kip/ft^2) for the primary energy application and an additional 0.36 MJ/m^2 (24 ft-kip/ft^2) during the ironing pass which is approximately the same energy previously calculated using Table 3.

Ground Improvement

In Figure 22, 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 ft.) and that the SPT values increased significantly.

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 23.

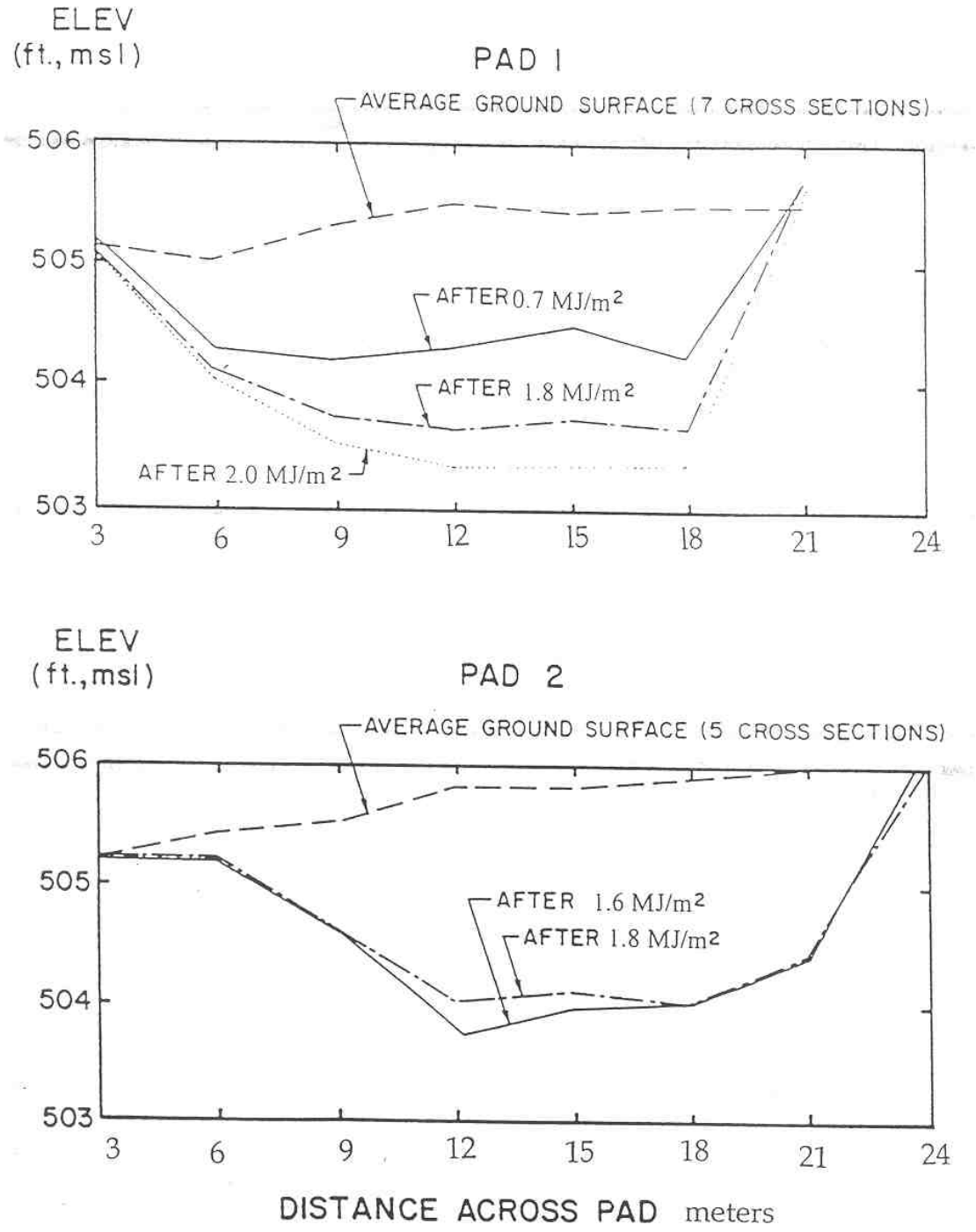

Figure 23. Induced Settlement Following Dynamic Compaction at the Mine Spoil Project in Alabama.

At test pad 1, the average ground subsidence following full application of the primary energy was approximately 0.6 m (2 ft.). At test pad 2, approximately 0.6 m (2 ft.) 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 ft.), 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. Figure 24 illustrates the variation in the crater depths that were observed in certain sections of the dynamically compacted area. The normal average crater depth for the mine spoil was 1-1.1 m (3.3-3.6 ft.), but crater depth as high as 1.5-2.7 m (5-9 ft.) 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.

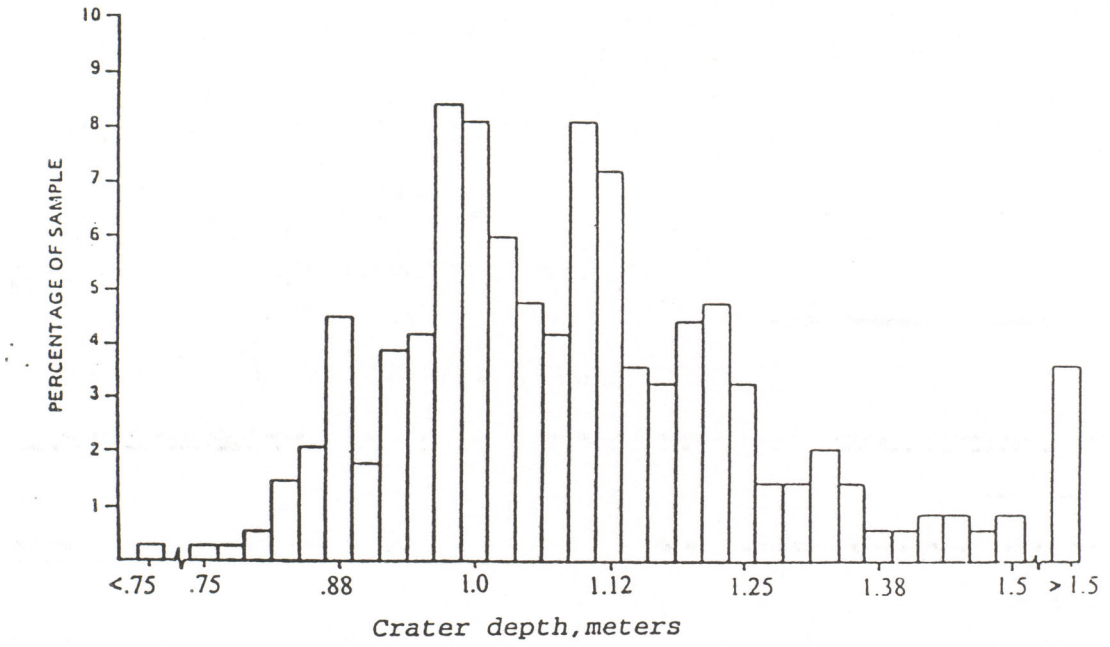

Figure 24. Statistical variation in crater depths at the mine spoil site in Alabama.

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 ft.) of soil followed by placement in lifts and compaction would have been approximately 2.6 times the cost of the dynamic compaction.

7.2 HIGHWAY EMBANKMENT ON LANDFILL DEBRIS

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 ft.) long, and the planned embankment extended to a height of approximately 7 m (23 ft.) above present grade. The northerly area, designated as area 2, is approximately 91 m (300 ft.) long, and the new planned embankment extended to heights of 3-3.7 m (10-12 ft.) above present grade.

Subsurface Conditions

Both landfills were covered with about 0.6 m (2 ft.) of gravelly glacial till that was used as a cover material. Below this level, old landfill material was present to depths ranging from 1-3.4 m (3.3-11.2 ft.), but averaging about 2 m (6.6 ft.) 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 10-14 blows for 300 mm (1 ft.), with some values as low as 7.

At area 2, the thickness of the trash was approximately 1-6 m (3.3-20 ft.) and averaged 3 m (10 ft.). 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 ft.) 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.

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.

Predicted Densification Procedure

Using the guidelines in this technical summary, 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 ft.) maximum in area 1, 6 m (20 ft.) maximum in area 2.

For area 2:

Using Equation 1, with n = 0.4 for a landfill, and D = 6 m

$$6 \text{ m} = 0.4 \, (WH)^{1/2}$$

$$WH = 225 \text{ Mgm}$$

From Figure 20, the desired energy can be obtained as the product of tampers ranging from about 13-16 Mg multiplied by drop heights ranging from 14-30 m.

For a 14-Mg tamper, use:

$$H = \frac{225\,Mg\,m}{14\,Mg} = 16\,m$$

For area 1:
Using Equation 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 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 ft.), suggesting a grid spacing of 2.3 m. 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 2.

For area 2, a grid spacing of 2.3 m, and 1 pass

$$2400\,kJ/m^2 = \frac{(14\,Mg)(10\,kN/Mg)(16\,m)(N)(1)}{(2.3\,m)^2}$$

N = 5.66 drops or 6 drop per grid point.

For area 1, a grid spacing of 2.3 m, and 1 pass

$$1600\,kJ/m^2 = \frac{(14\,Mg)(10\,kN/Mg)(5.2\,m)(N)(1)}{(2.3\,m)^2}$$

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:

$$1600\,kJ/m^2 = \frac{(14\,Mg)(10\,kN/Mg)(H)(6)(1)}{(2.3\,m)^2}$$

$$H = 10\text{ m}$$

After dynamic compaction, the maximum SPT value according to Table 2 would be anticipated to be 15-40.

The estimated induced settlement would be (20%)(3 m) = 0.6 m (1 ft.) in area 2, and (20%)(2 m) = 0.4 m (1.3 ft.) in area 1, based on experience with landfills.

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 (2ft.)blanket of silty sandy gravel was placed on the surface as a working mat.

The specifications required a 13.6-Mg (30 kips) (minimum) tamper, an 18-m (60 ft.) drop in the shallow fill area, and a 27-m (90 ft.) drop height in the deeper fill area. The contractor used a 14-Mg (31 kips) tamper with 18 and 27 m (60 and 90 ft.) 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 ft.) between drop points. The second phase consisted of the same grid pattern, offset from the first by 2.3 m (7.5 ft.) 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-kip/ft^2) for area 1 and 3.75 MJ/m^2 (254 ft-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 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 ft.). In the third phase, which took place at the same location as the first phase, the crater depths were 0.5 m (1.6 ft.). Heave measurements were taken

adjacent to the drop point locations, and heave was not observed. Figure 10 is a job photo illustrating the dynamic compaction equipment and the grid spacing, plus crater depths.

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.

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 ft.) in the shallow fill areas to 27 m (88.6 ft.) 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 ft.) gravel blanket used as a working mat.

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Technical Summary # 5:

STONE COLUMNS

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CHAPTER 1
DESCRIPTION AND HISTORY

Over the past 25 years, stone column technology has become established in the United States as a viable ground improvement tool. 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. Stone columns have a proven record of experience and are ideally suited for improving clays, silts, and loose silty sands. The history and development of stone columns and the methods of stone column construction are discussed in this chapter. In addition, this chapter introduces vibro-concrete columns (VCC), an extension of the bottom-feed vibro technique, geotextile-encased columns (GEC), and Geopier® Rammed Aggregate Pier™ an intermediate depth ground improvement system installed by tamping stone in a preformed cavity.

1.1 DESCRIPTION

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 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 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.

Vibro displacement 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.

1.2 FOCUS AND SCOPE

The focus and scope of this technical summary on stone columns and ancillary technologies (*i.e.,* VCC, GEC, and rammed aggregate piers) is to identify problems that have been successfully solved by the stone column method and to synthesize the current state-of-the-practice of stone column construction and design. 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 stone columns, VCC, GEC, and rammed aggregate piers can solve a specific problem. It is the intent of this document to serve as a reference on stone columns and ancillary technologies and how they may solve a specific problem by discussing their construction, utilization, and limitations.

1.3 RECOMMENDED READING

Federal Highway Administration *"Design and Construction of Stone Columns Vol. 1"* was the first complete source of technical data and specifications for this technology in the United States.[1] Portions of this technical report will be updated by this technical summary. Since the publication of this technical report, the design and construction industry's general acceptance of stone columns has occurred, and a large number of projects have been successfully completed.

A number of technical papers are now available that detail developments in the technique and discuss significant applications. The following publications are recommended:

- ***Deep Foundation Improvements: Design, Construction and Testing*** Contains 22 papers presented at the symposium on Design, Construction and Testing of Deep Foundation Improvements Stone Columns and Other Related Techniques.[2] The majority of these papers discussed stone columns.

- ***Soil Improvement - A Ten Year Update, (1987)***[18] ASCE Special Geotechnical Publication No. 12. Extensive discussions on stone column design.

- ***Ground Improvement/Reinforcement/Treatment Developments 1987-1997***[3] ASCE Special Geotechnical Publication No. 69. Extensive discussion on the use of stone columns for liquefaction mitigation.

- ***Soil Improvement For Earthquake Hazard Mitigation***[4] Summarizes the performance of improved soils at more than 30 locations during 7 large earthquakes, beginning with the 1964 Niggata quake and culminating with the 1995 Kobe quake.

1.4 HISTORICAL OVERVIEW

For over 50 years, depth vibrators have been used to improve the bearing capacity 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 18 meters 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 1. 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 zone are generally readily compacted by the process known as vibro-compaction. If the grain size distribution curve falls in the hatched zone, it is advisable to backfill with stone in lieu of sand during the compaction process to improve the contact between the vibrator and the treated soil. The many other soils with grain size distribution curves partly or entirely on the fine side of the hatched zone are not readily compacted by vibro-compaction. It is for these types of soils and their related problems that necessitated the development of stone column technology.

It is interesting to note that the first documented use 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.[1] 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 over 20 times, 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.

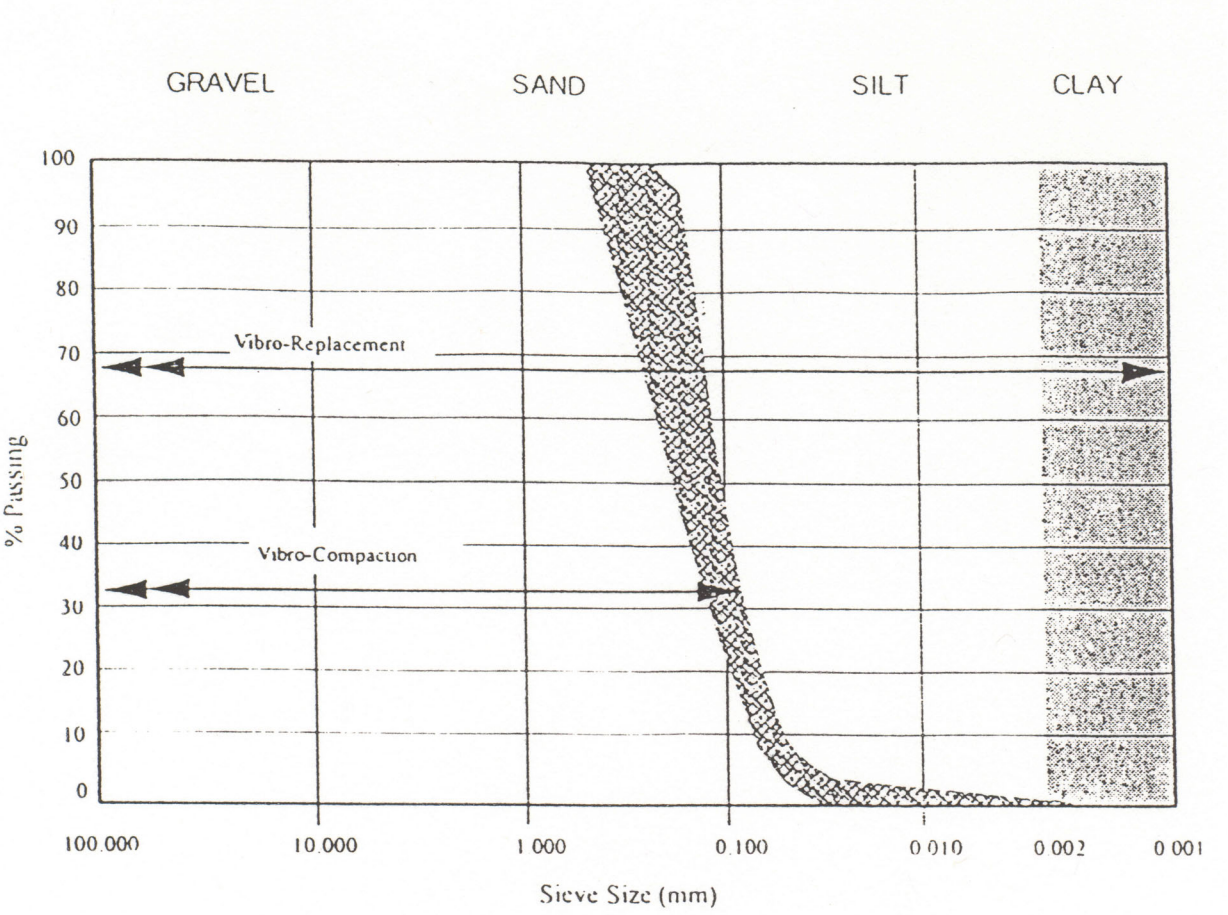


Figure 1. Grain Size Distribution Curves.

Much of the information and technology utilized today in the United States has been gained through European practice and research. A report similar to *"Design and Construction of Stone Columns,"* entitled *"Lessons Learned from European Practice on the Use of Stone Columns for Site Improvement,"* describes the development and improvement of vibro technology.(5)

1.5 TYPES OF STONE COLUMN CONSTRUCTION

The primary methods of constructing stone columns are vibro-replacement (wet, top feed) and vibro-displacement (dry, top or bottom feed). Figures 2 and 3 illustrate these processes. Although the dry, top and bottom feed methods have been used on many projects, the wet method is more typically used today. VCC, GEC, and rammed aggregate piers may be considered as emerging technologies.

Where environmental concerns are strong, the dry process will typically be required. However, the wet process is more economical, if environmental concerns are not relevant to the project. GEC construction is a dry-construction method that may use either the displacement or replacement technique.

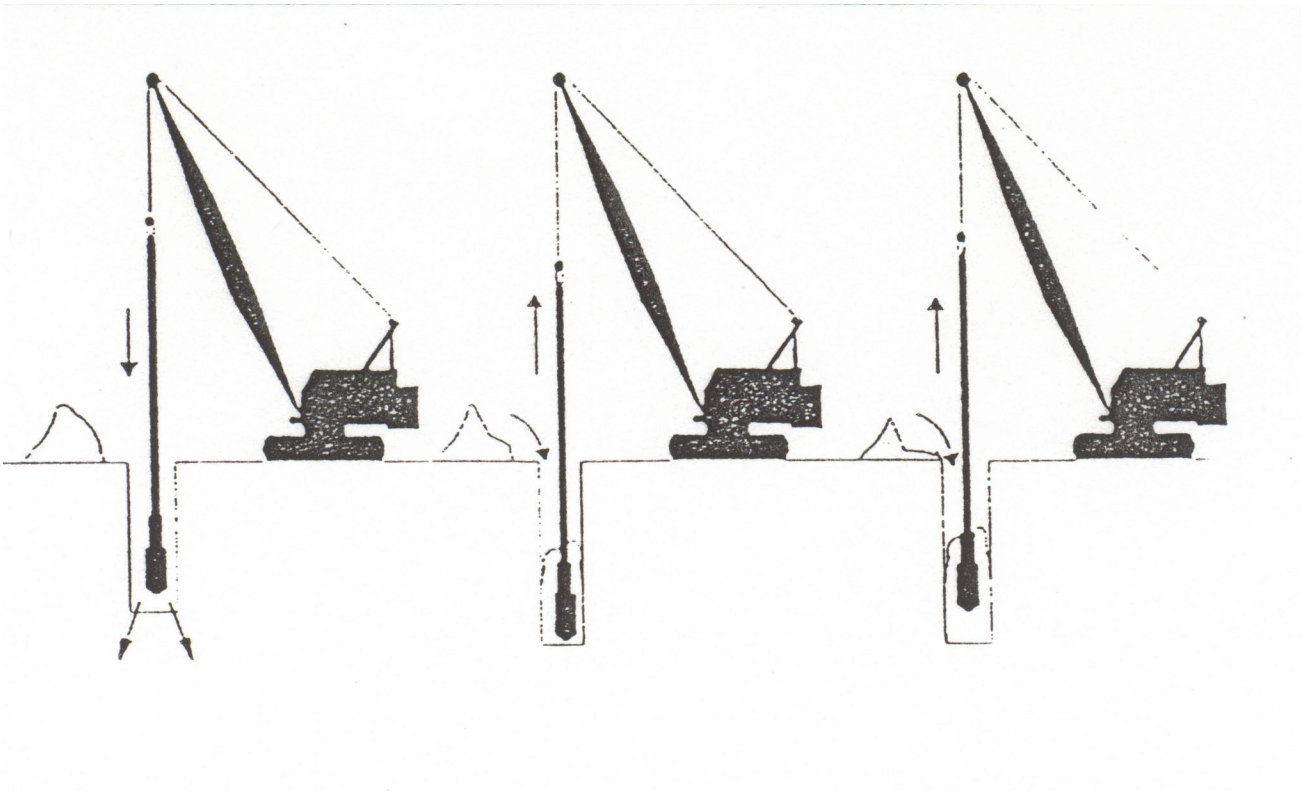


Figure 2. Top Feed Vibro-Replacement.

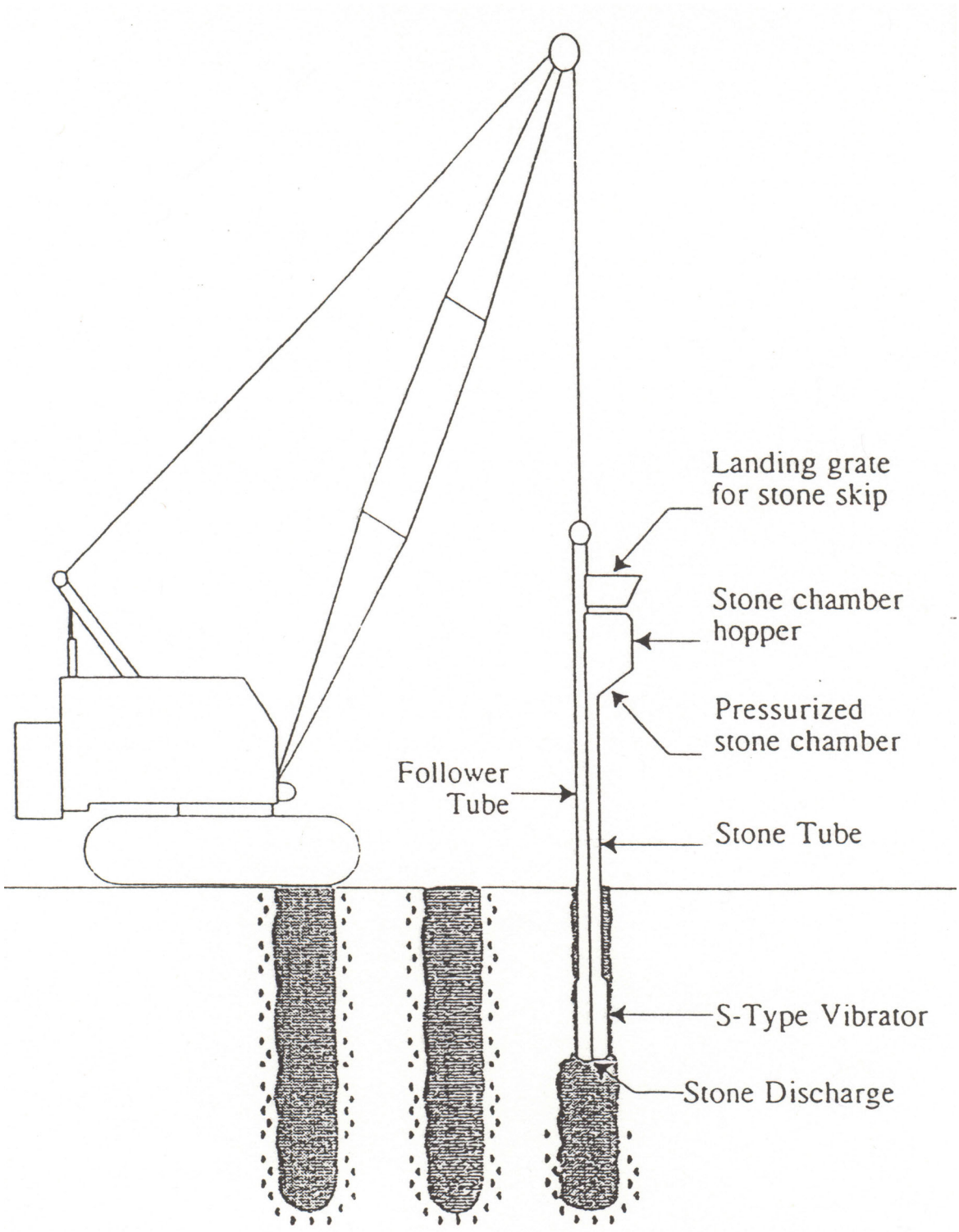


Figure 3. Bottom Feed Vibro Displacement.

a. 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 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 10-50 kN/m^2 (200-1000 psf) and a high groundwater table.

b. 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 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 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.[6]

c. Vibro-Concrete Columns

The vibro-concrete column was first developed in Europe in 1976. Since stone columns derive their strength and settlement characteristics from the surrounding soil, they do not perform well in very soft clay or peat with a thickness greater than 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 improvement 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 Philadelphia, PA, and was used in support of an oil storage tank.[7] They have been used to support embankments over soft organic soils in New Jersey, South Carolina, and Puerto Rico.

The vibro-concrete column process employs a bottom feed depth vibrator that can penetrate the soils to a level suitable for bearing. Concrete is pumped through the vibrator assembly during initial withdrawal. The vibrator then repenetrates the concrete, displacing it into the surrounding soil to form a high-capacity, enlarged column base. The vibrator is then slowly withdrawn as concrete is pumped at 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 loading into the vibro-concrete column (Figure 4).

d. Geotextile-Encased Columns

Geotextile-encased columns (GEC) were developed in Europe by a German contractor (Mobius) and patented in 2001. GEC consists of inserting continuous, seamless, high strength geotextile tubes into soft soil with a mandrel, and filling the tube with sand (or fine gravel) to form a column with a high bearing capacity. The typical diameter of a GEC is 0.8 m (2.6 ft.). There are two installation methods for GEC.

Replacement Method – an open end steel pipe is driven to the bearing stratum. Once the desired depth is reached, the soil inside the casing is removed with an auger. The geotextile is then inserted into the casing and filled with sand.

Displacement Method – a steel pipe with two base flaps (which close upon contact with the soil) is vibrated down to the bearing layer, displacing the soft soil. The geotextile casing is installed and filled with sand. The casing is extracted by vibration. During this removal process, the sand within the geotextile is densified.

e. Geopier® Rammed Aggregate Piers™

The concept of short dug pits backfilled with aggregate to support structures is not new and has been previously documented in the literature.(1) Refinements and improvement to this basic technique have been introduced in the last decade under the trade name Geopier®. Geopier rammed aggregate piers are a ground improvement system that modifies the soil through the installation process. As shown in Figure 5, rammed aggregate piers are installed by drilling 0.6-0.9 m (2-3 ft.) diameter holes into the foundation soils and ramming thin 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 2-10 m (6.6-33 ft.) below grade.

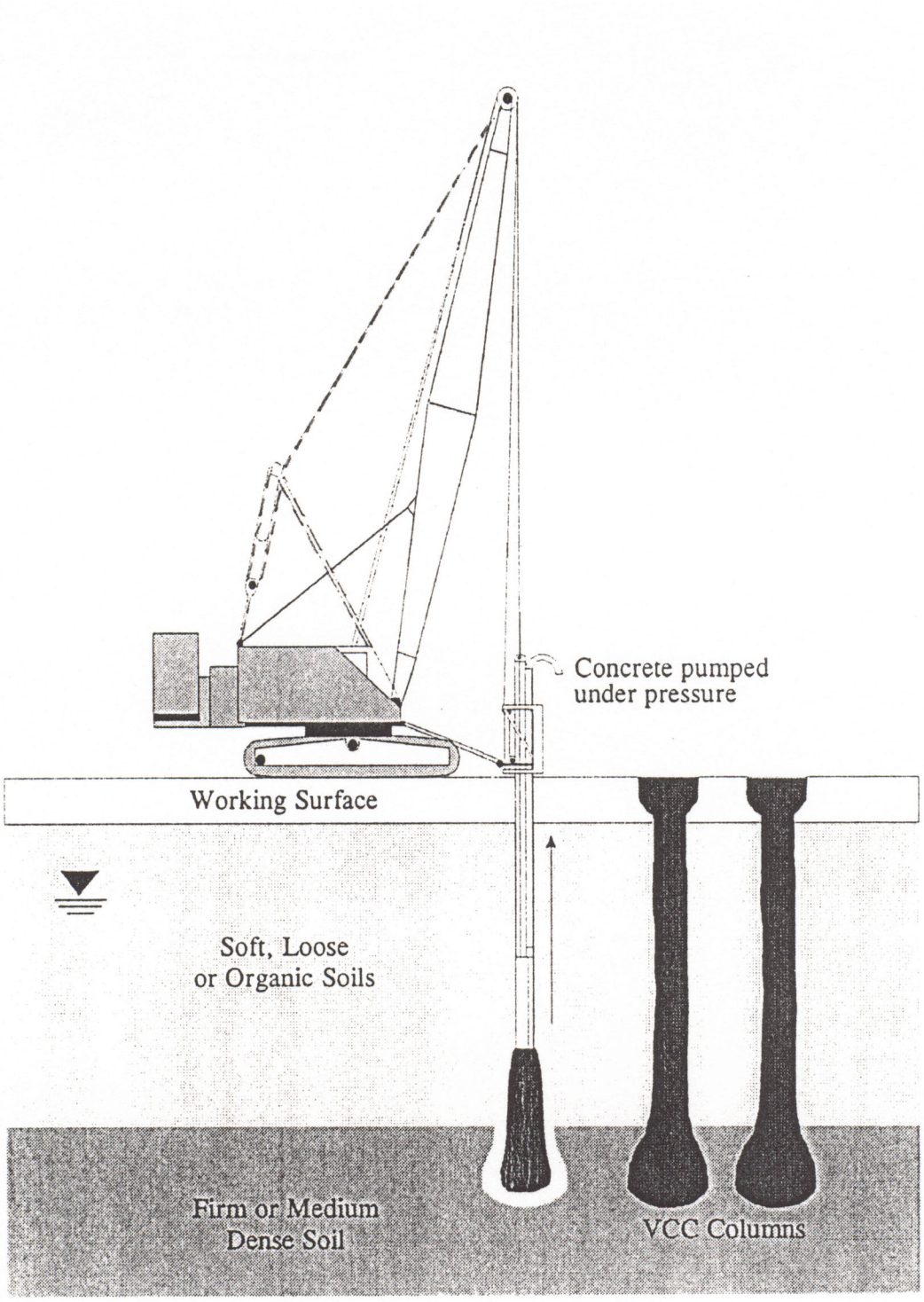


Figure 4. Vibro-Concrete Columns.

The first lift of aggregate forms a bulb below the bottoms of the piers, thereby pre-stressing and pre-straining the soil at the bottom of the pier. Subsequent lifts of aggregate are typically 0.3 m (1 ft.) in thickness. Ramming takes place with a high-energy beveled tamper, with an impacting energy from 34,000-90,000 kg-m per 0.3 m (245-650 kip-lbf per 1 ft.) lift, 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.[13,15]

The rammed aggregate pier method is most suitable for soft to stiff cohesive soils with undrained shear strengths ranging from 15 kN/m^2 to 180 kN/m^2, (310-3800 psf) and loose to medium dense silty and clayey sand.

This rammed aggregate pier construction equipment and the process of making the aggregate pier is subject to a United States patent which was issued in 1993.[14] The most economical application for this type of construction appears to be in connection with the support of structures or retaining walls on relatively shallow compressible foundations.

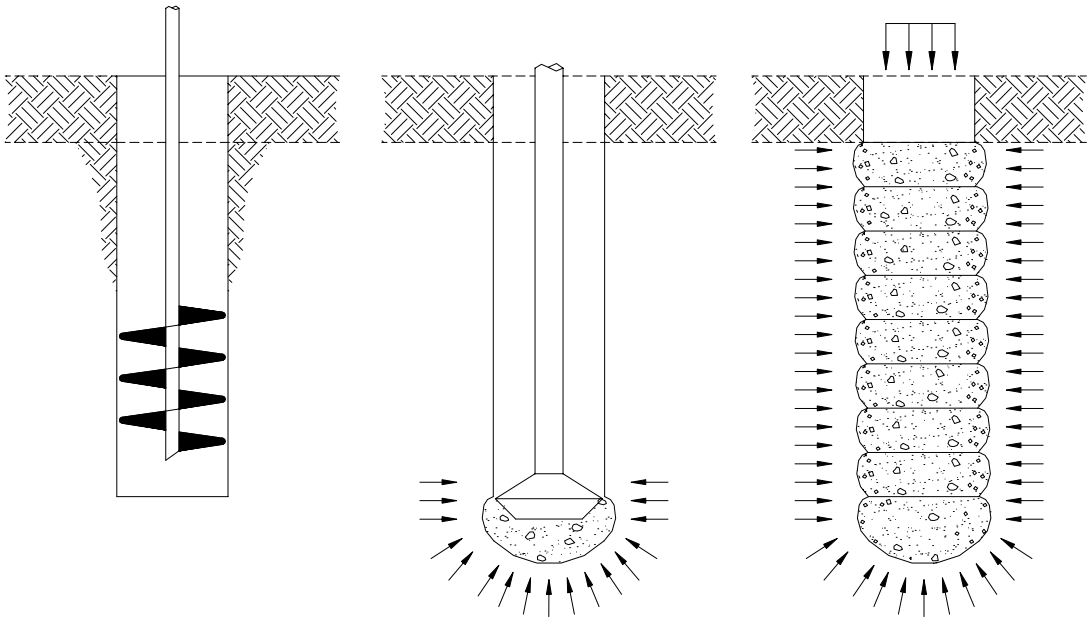


Figure 5. Geopier Rammed Aggregate Pier Construction Process.

CHAPTER 2
DESIGN CONSIDERATIONS

Stone column construction involves the partial replacement or displacement of unsuitable subsurface soils with a compacted vertical column. This chapter discusses applications, advantages and disadvantages, and design considerations for stone column technology.

2.1 APPLICATIONS

The technique can be applied to improving slope stability, increasing bearing capacity, reducing total and differential settlements, reducing the liquefaction potential of soil, and accelerating the time rate of settlement. Typical applications include foundation improvement for the construction of highways, embankments, warehouses, and light industrial buildings. Rammed aggregate piers have been used extensively for foundation support of buildings up to 18 stories tall.

a. Embankments

One typical application of stone column technology is the stabilization of large area loads such as highway embankments. The use of stone 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.[8]

A considerable amount of highway widening and reconstruction work will be required in future years. Some of this work will involve 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 the new fill on stone columns offers a viable design alternative to conventional construction.

b. Bridge Approach Fills

Stone 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 4,000 m (13,000 ft.) of stone columns to support mechanically stabilized earth for the U.S. 77 overpass situated in Brownsville, TX. In 1990, the Texas DOT utilized 12,800 m (42,000 ft.) of 4-6-m-deep (13-20 ft.) stone columns for Brownsville road over U.S. 77.[9] VCCs have been used adjacent to pile supported bridges with stone columns farther away from the bridge structure to reduce the potential for differential settlement and "the bump" at the end of the bridge.

Under favorable conditions, stone 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 stone columns. Embankment fills can be placed faster due to the combined effects of accelerated drainage and consolidation increase in shear strength supplied by stone columns.

c. Bridge Abutment and Foundation Support

Stone 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 stone columns. This technique minimizes the differential settlement between the bridge and approach fill.

d. Liquefaction

In earthquake prone areas, stone columns can be used to reduce the liquefaction potential of cohesionless soils supporting embankments, abutments, and soils beneath shallow foundations. Stone 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 DISADVANTAGES OF STONE COLUMNS, GEC AND RAMMED AGGREGATE PIERS

Advantages

Stone columns are a technical and potentially economical alternative to deep foundations, capable of improving the soil sufficiently to allow less expensive, shallow-foundation construction. Stone 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 dynamic compaction, deep blasting or piling, the low-vibration stone column technique is often viable. If time is critical to project start-up, site improvement by stone column installation can be achieved quicker than by pre-loading the soils. In seismic areas, stone columns can densify the soils beyond the threshold of liquefaction. Stone columns also provide a vertical drainage path for excess pore water pressure dissipation, as well as densifying the soils.

Vibro-concrete columns use the load transferring characteristics of piles, while mobilizing the full ground improvement potential of the vibro systems. Construction of the columns is a very quiet process, with minimal surface vibration, allowing work close to structures. As it is a dry process, no spoil removal is required. Due to the enlarged-base construction, column lengths are shorter than would be required for conventional piles. Where thick strata of very soft clay or organic material such as peat are present, vibro-concrete columns can also be a technically feasible and economical solution.

The major advantage of GEC over stone columns is that they may be used in soft soils with undrained shear strengths as low as 0.4 kN/m^2 (25 psf). The geotextile provides the lateral constraint that the surround soils must provide for a stone column. GEC provide excellent vertical drainage, which may result in very rapid construction, due to the dissipation of porewater pressure. Quality assurance/quality control may be easier to perform on GEC than on stone columns.

Rammed aggregate piers provide an alternate construction technique with engineering characteristics similar to stone columns as a result of the direct ramming of thin lifts of aggregate. They may be used when an "intermediate" depth unit is technically feasible. "Intermediate" depth refers to a project specific pier length, which is technically necessary to provide the required structural support with acceptable settlement and is economical when compared to alternate solutions. Most applications involve depths ranging from less than 5 m (16 ft.) up to 10 m (33 ft.) using larger equipment and tampers. Rammed aggregate piers are used in a variety of soil conditions to support buildings, retaining walls, and

embankments. The rapid installation process often results in time and cost savings compared to other technologies.

Disadvantages

Stone columns are not a solution for all soft soil problems. Strata of peat, mulch, and other organic materials, and very soft clays with a thickness greater than the diameter of the stone column can be inappropriate for stone column construction, as they offer inadequate lateral support. Where very soft strata are present to the depth of improvement, stone column construction after replacement and/or displacement of the very compressible material can still be the most economical option. One site for a 14-story structure had a 6-meter stratum of peat under a portion of the proposed foundation. The peat was removed and the zone backfilled with end-dumped granular fill. Both the fill and the existing loose soil were then densified by stone columns.[10] Dense overburden, boulders, cobbles, or other obstructions may require pre-drilling prior to installation of stone columns.

Cost, when compared to other solutions, can be a major disadvantage of stone 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.

The major disadvantage of GEC and rammed aggregate piers is that they are both proprietary, patented technologies. GEC have a limited track record. Rammed aggregate piers are limited to a depth of approximately 10 m (33 ft.) and are difficult to install in clean sands when the groundwater table is above the bottom of the pier. Rammed aggregate piers are more costly to install when casing is required and do not densify granular soils as effectively as stone columns.

2.3 FEASIBILITY EVALUATIONS

The stone column technique of ground treatment has been successful in (1) improving stability of both embankments and natural slopes, (2) increasing bearing capacity, (3) reducing total and differential settlements, (4) reducing the liquefaction potential of cohesionless soils, and (5) increasing the time rate of settlement.

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, stone column area, column spacing, and compaction energy applied. Experience has shown that soils with less than 15 percent passing a 0.074

mm (#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 displaced by the stone column, VCC, GEC, or Geopier.

A generalized summary of the factors affecting the feasibility of stabilizing soft ground with stone columns is as follows:

1. The allowable design loading of a stone column should be relatively uniform and limited to a maximum of 500 kN (110 kips) per column, if sufficient lateral support by the in-situ soil can be developed.

2. The most significant improvement is likely to be obtained in compressible silts and clays occurring within 10 m (33 ft.) of the surface and ranging in shear strength from 15-50 kN/m^2 (300-1000 psf).

3. Stone columns should not be used in highly sensitive soils. Special care must be taken when using stone columns in soils containing organics and peat lenses or layers with undrained shear strength of less than 10 kN/m^2 (200 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 1 to 2 stone column diameters, the ability to develop consistent column diameters becomes questionable.

4. Ground improved with stone columns reduces settlements typically from 30-50 percent of the unimproved ground response and differential settlements from 5-15 percent of unimproved soil response.

5. Stone columns have been used in clays having minimum (not average) undrained shear strengths as low as 7 kN/m^2 (150 psf). Due to the development of excessive resistance to penetration of the vibrator and economic considerations, a practical upper limit is in the range of an undrained strength of 50-100 kN/m^2 (1000-2000 psf). Clays with greater shear strengths may, in fact, be strong enough to withstand the loads without ground improvement. If stone columns are used in these stiff soils or through stiff lenses, the column hole is commonly prebored, which is often the case in landslide projects. This situation will result in a significant additional cost.

6. Individual stone columns are typically designed for a bearing load of 180-270 kN (20-30 tons) per column. The ultimate capacity of a group of stone columns is predicted by estimating the ultimate capacity of a single column and multiplying that capacity by the number of columns in the group.

7. Stone columns have been used effectively to improve the stability of slopes and embankments. The design is usually based on conventional slip circle or wedge analyses utilizing composite shear strengths.

A generalized summary of the factors affecting the feasibility of stabilizing soft ground with VCC follows:

1. The allowable design load for VCC 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 450-600 mm (18-24 in.). Typical allowable design loads for VCC range from 650-900 kN (75-100 tons).

2. VCC are typically used in very soft clay and organic soils.

3. Typical VCC lengths vary from 5-10 m (16-33 ft.).

A generalized summary of the factors affecting the feasibility of stabilizing soft ground with GEC follows:

1. GEC may be installed in soft, compressible clays up to depths of approximately 10 m (33 ft.). Typical column diameters range from 0.7-0.9 m (2.3-3.0 ft.).

2. GEC allowable load capacity is 175-350 kN (20-40 tons).

3. Settlement of GEC typically occurs during construction of embankment and may be up to 250-500 mm (10-20 in.).

A generalized summary of the factors affecting the feasibility of stabilizing soft ground with Geopiers follows:

1. The allowable design load for Geopier rammed aggregate piers is typically in the range of 225-650 kN (25-75 tons) per pier, depending on the lateral confinement provided by the surrounding soil (*i.e.,* undrained shear strength $\geq$ 15 kN/m^2 (300 psf) for soft saturated clays, SPT N-value $\geq$ 1 for cohesionless soils).

2. Geopier rammed aggregate piers have been used effectively in soft soils, provided that top-of-pier stresses are lower than those needed to initiate pier bulging into the soft soils.

3. The installation of Geopier rammed aggregate piers in soils that do not stand open during drilling (loose granular soils, very soft cohesive soils) often requires the use of temporary casing, which reduces the installation rate and increases the cost of the piers.

4. The maximum practical depth of Geopier rammed aggregate piers is limited to 10 meters.

a. Improvement of Settlement Characteristics

Stone columns, CEGs, and rammed aggregate piers act similarly to prefabricated, vertical drains (wicks) in decreasing the distance in which water has to flow in the radial direction for primary consolidation to occur. As a result, installation of stone columns, GECs, and rammed aggregate piers can, in the absence of natural drainage layers within cohesive soils, significantly decrease the time required for primary consolidation. Under these conditions, the presence of stone columns, GECs, and rammed aggregate piers will greatly accelerate the gain in shear strength of cohesive soils as primary consolidation occurs. In addition, total settlement is reduced as the columns carry a greater portion of the total surface load.

Laboratory consolidation tests should be performed to evaluate the vertical and horizontal consolidation properties of the compressible stratum. A careful examination of the undisturbed samples index property tests and site geology can also be used as a guide in estimating the ratio between horizontal and vertical rates of consolidation.

On some projects, reliable estimates of time rate of settlement are necessary for the success of the project or for a reliable comparison of design alternatives. For such projects, the horizontal permeability may be evaluated using field pump tests. Piezometer or well point permeability tests are alternatives that should yield horizontal permeabilities equal to or less than those obtained from pump tests. If the vibro-replacement method is to be used, the drains and well points should be installed by jetting.

Secondary settlement equal to or even greater than primary consolidation settlement can occur in highly organic soils and highly plastic, inorganic clays; significant secondary settlement can also occur in highly micaceous soils. These soils are likely candidates for

reinforcement with stone columns, GECs, and rammed aggregate piers to support embankment loads. Secondary settlement will therefore be an important consideration in many projects involving these ground improvement techniques. Because of the relatively short time usually required for primary consolidation to take place in column reinforced soils, secondary settlement is even more important than if columns are not used. However, stress concentration in the stone columns reduces the stress on the soil between columns, thus reducing the magnitude of secondary settlement. Neither stone columns, GECs, rammed aggregate piers, nor vertical drains affects the time rate for secondary settlement.

The magnitude of secondary compression can be reduced to tolerable levels by surcharge loading. The amount of secondary compression that occurs is directly related to the level of the surcharge and to be effective, the surcharge must apply an effective stress greater than that ultimately reached under the service loading. For sites where secondary settlement control is important, consideration should be given to surcharge loading, particularly at transition areas of small to large settlements.

b. Earthquake Liquefaction Mitigation[11,12]

One of the most dramatic causes of damage to structures during earthquakes has been soil liquefaction beneath and around these structures. The phenomenon is associated primarily, but not exclusively, with strong earthquake ground motion, relatively loose cohesionless soils, and a lack of drainage during the seismic event that leads to excess pore water pressure generation and reductions in effective stresses.

The effects of soil liquefaction have been observed for many years. When a saturated cohesionless deposit is subjected to strong shaking, it tends to compact and decrease in volume. However, if drainage is prevented, volume changes cannot occur and stresses are partially transferred to the pore water. The stress transfer causes a rise in pore water pressure and a corresponding drop in soil particle effective stress. If the effective stress becomes zero, contact is lost between soil grains and a liquefied state develops.

The purpose of a stone column design for this application is to mitigate the potential for liquefaction and minimize damaging settlement in the soil. Ways to achieve mitigation include

1) increasing the soil density,
2) increasing the confining pressure,
3) increasing the material stiffness by installing stone columns or rammed aggregate piers, or

4) increasing drainage for relief of excess pore water pressures.

These changes lead to control and/or prevention of pore pressure development and, thereby, reduce the likelihood of soil liquefaction.

The installation of stone columns mitigates the potential for liquefaction by increasing the density of the surrounding soil, providing drainage for the control of pore water pressures, and introducing a stiff element (stone column) that can potentially carry higher shear stress levels.[9] Rammed aggregate piers provide the same advantages, except that stone columns are much more effective at increasing the density of the surrounding soils.

Densification of loose soils is one of the most common and reliable methods of reducing earthquake-related liquefaction potential. However, liquefaction potential can also be minimized by controlling the level of earthquake-induced excess pore water pressures developed in the soil and by reducing the level of cyclic shear stresses experienced by the soil during strong ground motion.

Parameters affecting the stone column performance (earthquake-induced pore pressures and settlements) include ground motion characteristics, such as peak acceleration, frequency, duration, and the properties of the soil-stone column system. These soil-stone column properties include the degree of densification, shear moduli, compressibility, permeability, and the geometry of the system. Current liquefaction mitigation design approaches in the United States consider primarily the increase in soil density. The benefits of improved drainage and the stiffness of the stone columns are typically ignored in the analysis. However, in Japan stone columns that are installed without densification are designed to act as pore pressure dissipation sinks in the event of an earthquake.

Recent research has suggested that stone columns may be able to reduce liquefaction potential by reinforcement and seismic shear stress redistribution.[12] The reduction of shear stress is a function of the shear modulus ratio between the stone columns and soil, and the area replacement ratio. A mathematical model linking these variables has been developed and may be used to study feasibility.[12]

2.4 ENVIRONMENTAL CONSIDERATIONS

Vibro-replacement stone columns traditionally have been jetted in place, thus removing the finer portions of the influenced soil. The resulting fines-laden jetted water has to be temporarily contained to allow for sediment deposition and disposal. Jurisdictions have

varying regulations regarding the processes for these operations. Also, unknown contaminants may be removed and transferred to the environment by the jetting water. Due to its dry method of installation, the vibro-displacement method is a more environmentally friendly system, as the stone columns displace, rather than replace, the problem soil.

2.5 ALTERNATIVE IMPROVEMENT METHODS

In addition to rammed aggregate piers and GEC, the following alternative methods, which are similar in concept, have been used.

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.

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.

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 (similar to the Franki Pile and Geopier system). While this system can create some compaction of the surrounding soil, it is a very slow process (70 m/shift/rig) and, therefore, not economically competitive.

CHAPTER 3
CONSTRUCTION MATERIALS AND EQUIPMENT

3.1 MATERIALS

a. In-situ Soils

Project sites with mixed strata of granular and cohesive soils are excellent candidates for stone columns. In these profiles, stone columns act to densify granular soils and either replace or displace cohesive soils.

Usually, ground improvement using vibro-compaction can be accomplished at one-half to one-third of the cost of stone columns, because sand is used as a backfill material. Therefore, the vibro-compaction technique is preferred where it is applicable for densification of granular soils. The companion technical summary, "Vibro-Compaction," indicates that for effective use, the upper limit of silts is 15 percent, with clay less than 2 percent.

As previously stated, stone columns should not be considered where thick strata of peat or other organic materials exist or with soils with undrained shear strengths under 10 kN/m^2 (200 psf). VCC and GEC may be effective solutions when constructed in very low strength soil types, and rammed aggregate piers are effective in soft to stiff cohesive soils and silty and clayey cohesionless soils that extend to a depth of 10 m (33 ft.) below the ground surface, provided that the applied top of pier stress is lower than the stress required to initiate bulging.

b. Backfill Material

The size and shape of backfill for stone columns usually depends on the construction technique. For vibro-replacement stone columns, rounded or subangular gravel of nearly uniform grading (25 mm to 60 mm (1.0 in. to 2.4 in.)) is used. This sized 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 increases the stability of the jetted hole.

For vibro-displacement, well graded backfill with 10-mm to 100-mm (3/8 to 4-in.) gradation is preferred to allow the best 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 10 mm to 35 mm (0.4 in. to 1.4 in.) in size to avoid blockage of the equipment.

For rammed aggregate pier construction, clean 50-75 mm (2-3 in.) 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-grade base course aggregate is used.

3.2 EQUIPMENT[16]

The equipment used to form stone columns is comprised of the following:

- the vibrator, which is elastically suspended from extension tubes with air or water jetting systems (Figure 6)
- the crane or base machine, which supports the vibrator and extension tubes
- the stone delivery system
- the control and verification devices

The principal piece of equipment used to achieve compaction is the vibrator, which ranges in diameter from 300-460 mm (12-16 in.) and in length from approximately 3-5 m (10-16 ft.). A cross section of a typical vibrator is shown in Figure 7.

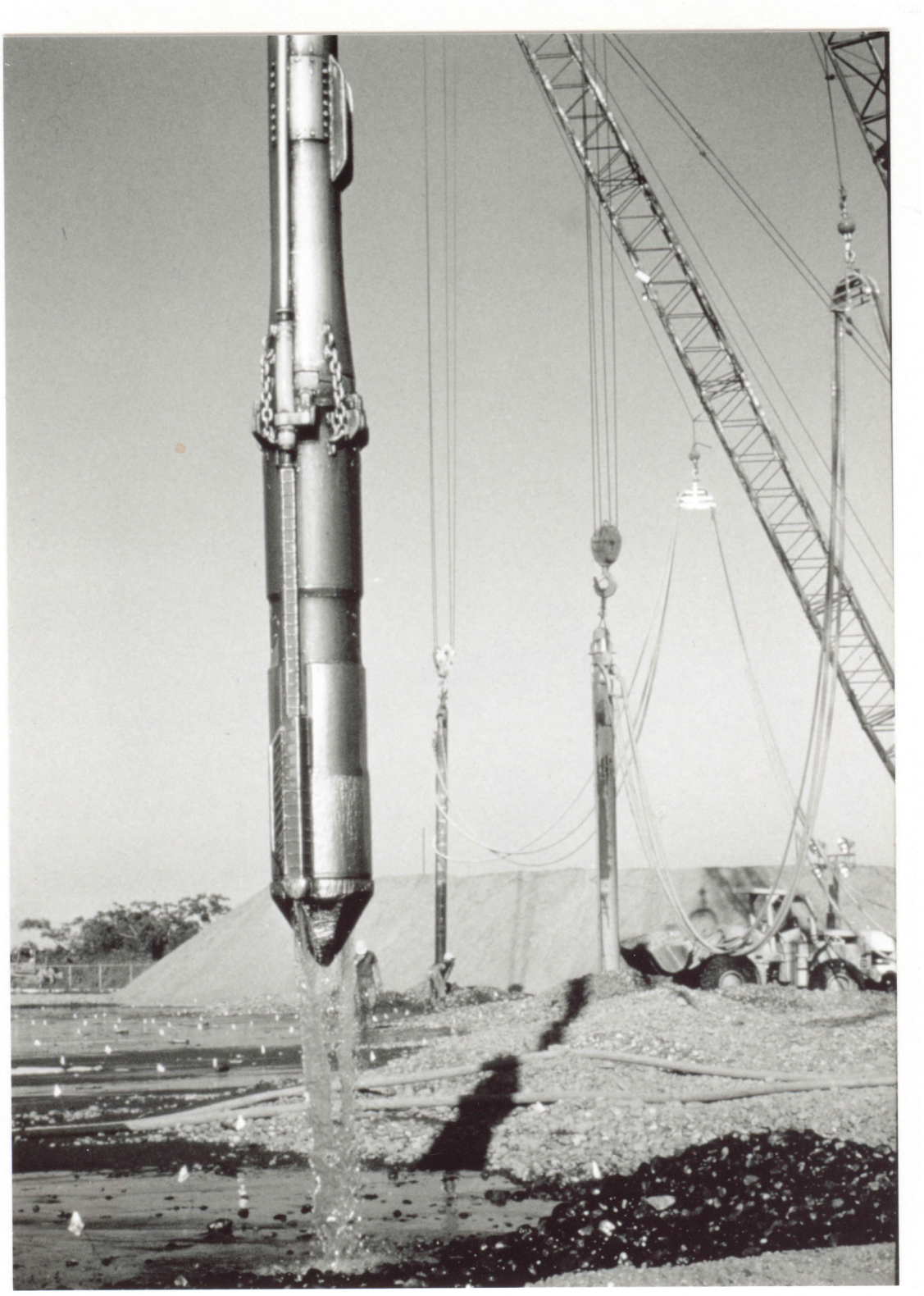


Figure 6. Elastically Suspended Vibrator.

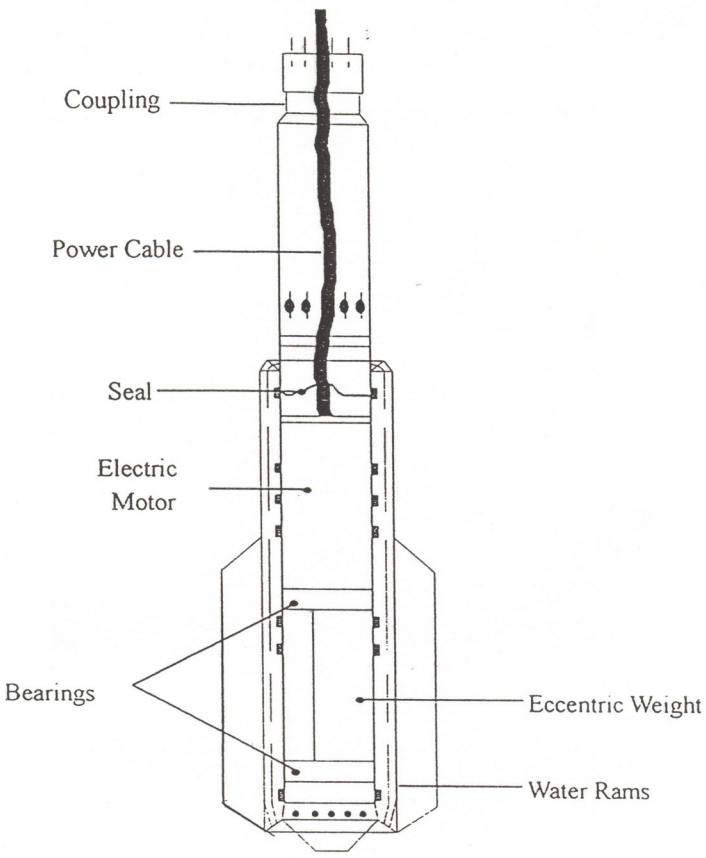


Figure 7. Vibrator Cross-Section.

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 22-60 kW range, but more recent machines develop up to 125 kW. Centrifugal forces of up to 27 Mg (30 tons) at frequencies varying from 1200-3000 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 from vibration 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 10-m (33 ft.) probe can be easily handled using a 36,000 kg (40 ton) crane with a 12 m (40 ft.) 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 150 mm (6 in.) hose instead of a skip, along a leader. Typical equipment to install stone columns is illustrated in Figures 8 through 11. For low headroom applications or small sites a system has been developed where the vibroprobe is mounted to an excavator (Figure 10).

A new method, marine double-lock gravel pump technique, has been developed to deliver stone to the bottom of the vibrator in underwater applications (Figure 8). This method transports the gravel through a system of hoses using air pressure supplied through an air compressor. The double-lock system provides at the tip of the vibroprobe excess air pressure at all times. This minimizes the potential for soil intrusion into the discharge pipe.

Recent development includes an instrumentation package that provides a continuous record of construction data for each stone column (Figure 12). 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.

The Marine Double-Lock Gravel Pump (Figure 8) has a Schnorchel hose (6), which is attached to a receiver tank (3). Schnorchel hose (6) and locks (4,5) are operated in such a way that during the gravel transport through hose (2) there is always atmospheric pressure in

the receiver tank (3). A high pressure compressor (8) feeds directly into the pressure tank (9), and, thus, (since either lock 4 or 5 is closed at all times) supplies a sufficient pressure to surmount the water and soil pressures in the gravel tube (10) at the tip of the vibroprobe (7).

The "Double Lock" assures that at the tip of the vibroprobe excess air pressure is applied at all times, avoiding the risk of clay intrusion into the gravel pipe and providing a very high discharge rate of stones into the ground. The patented "Schnorchel" hose allows air transport of the gravel against atmospheric pressures, although the receiver tank is submerged some 30 m, and possibly more, under water.

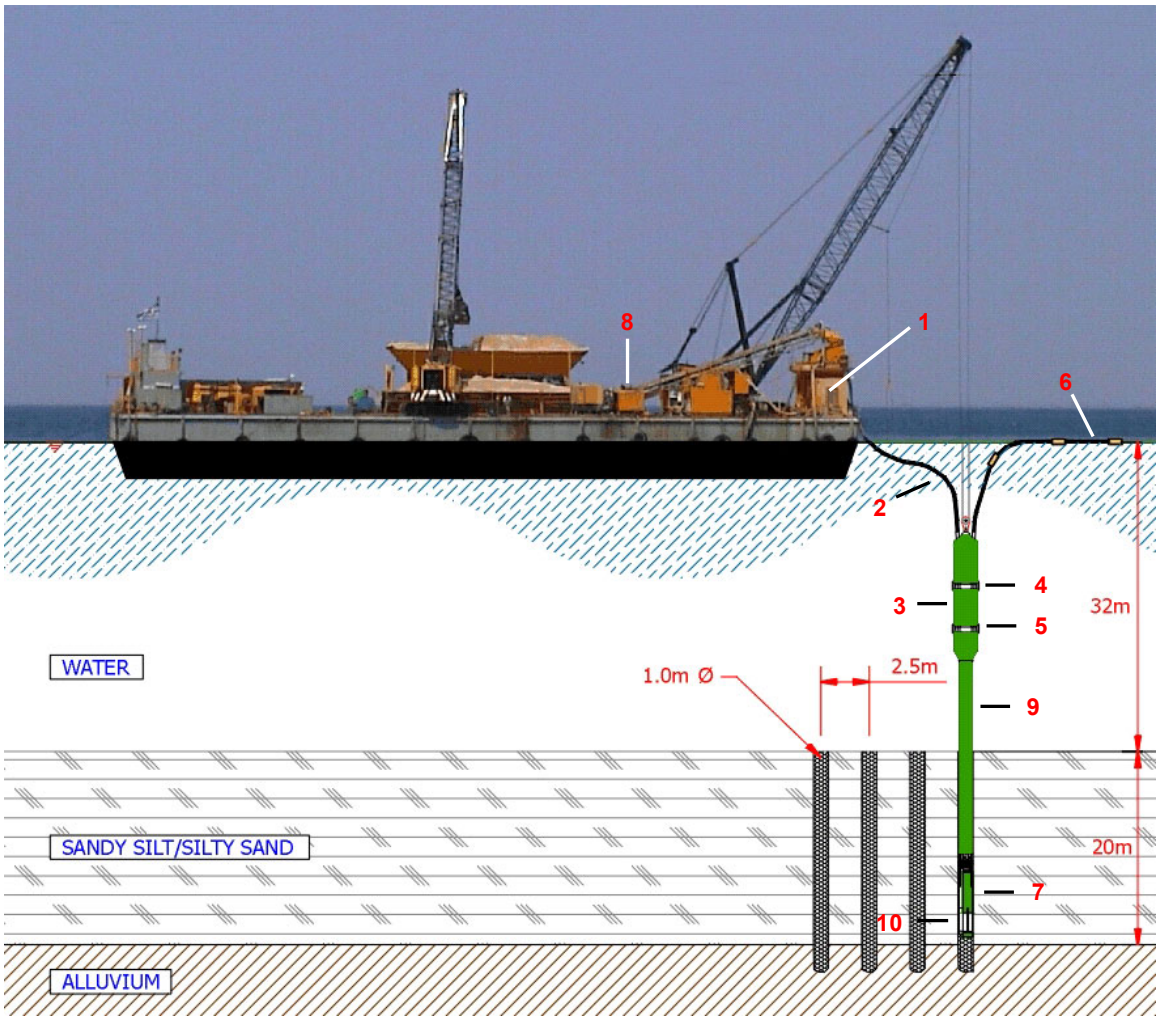
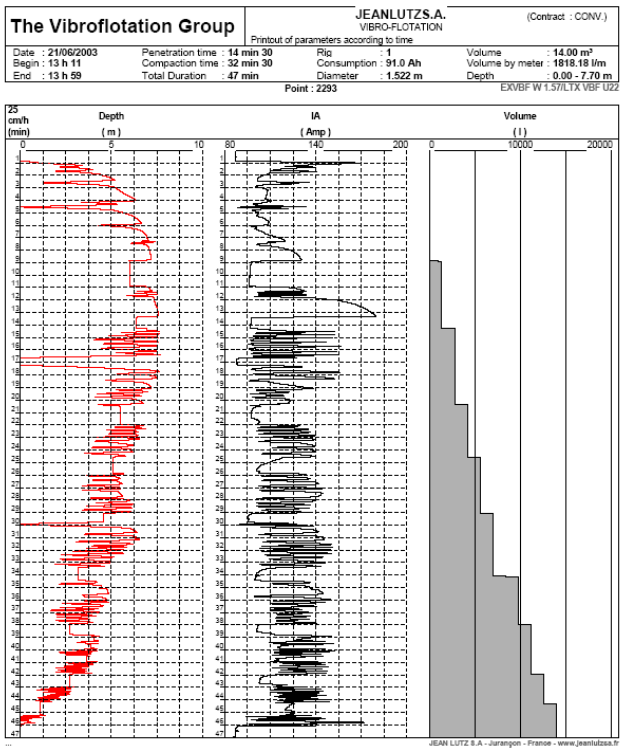


Figure 8. Marine Double-Lock Gravel Pump.

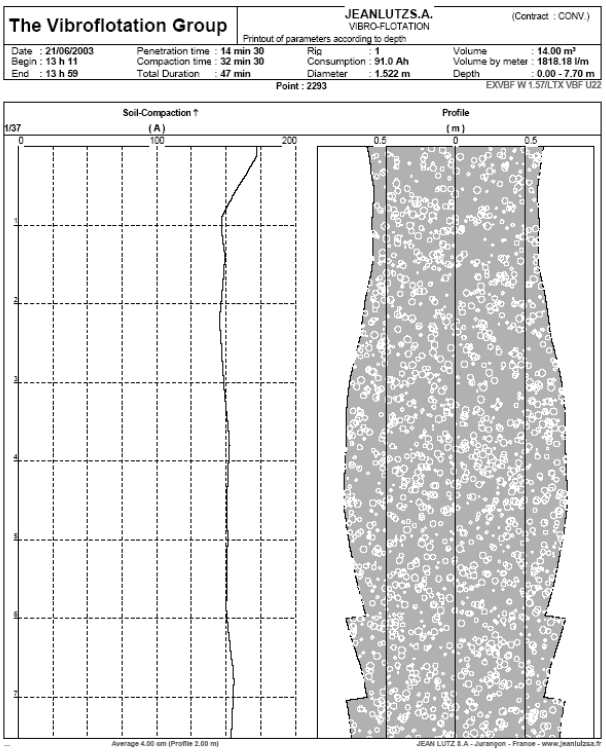
Figure 9. Truck Mounted Crane Utilized for Top Feed Vibro-Replacement.

Figure 10. "Vibro-Stitcher" - Excavator Mounted Dry Bottom Feed rig.[33]

Figure 11. Bottom Feed Vibro Rig with Stone Hopper.



OUTPUT OVER TIME
Depth (left), Amperage (middle) and Gravel placement (step curve on right)



OUTPUT OVER DEPTH
The same Stone Column, but output over Batch depth.
Left: Maximum Compaction Amperage
Right: Column Diameter over Depth

Figure 12. Quality Control Output – Dry Bottom Feed Vibro-Displacement.(33)

Rammed aggregate pier construction equipment is shown in Figures 13 and 14. 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 0.6-0.9 m (2-3 ft.) diameter drilling tools. Common excavators include a Caterpillar 315 in order to minimize problems and excessive 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. A composite alloy shaft with an attached beveled-hammer is connected to the modified concrete breaker. 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.

Figure 13. Rammed Aggregate Pier Tamper and Loader.

Figure 14. Rammed Aggregate Pier Equipment (Two Crews).

CHAPTER 4
DESIGN CONCEPTS

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, *"Design and Construction of Stone Columns–Vol I"* by FHWA can be consulted.[1] The most recent publication is *"The Design of Vibro Replacement"* by Priebe, which updates a widely used theory and method of calculation.[16] For Geopier design, the Geopier design manual may be consulted.[13]

More recently, "Ground Improvement, Ground Reinforcement, Ground Treatment: Developments (1987-1997)"[3] discusses design concepts to reduce liquefaction potential.

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 capacity, 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.

4.1 FUNDAMENTAL CONCEPTS

The fundamental concepts of stone column design are described in *"Design and Construction of Stone Columns"*[1] and *"In-Situ Ground Reinforcement"*.[18]

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.[19,20,21,22,23]

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 capacity 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.

For stability analysis, a composite shear strength of the soil stone-column is used along the sliding surface. The composite strength parameters are a function of the initial strength of the foundation soil, the frictional strength of the column soil, the area replacement ratio, and the stress ratio. The friction angle of the stone may be artificially increased to account for the stress concentration in the stone column when an embankment is constructed over the columns. The latter variables are defined below.

4.2 PRELIMINARY DESIGN

Stone columns are typically selected to increase bearing capacity, 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.

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 illustrated in Figures 15 and 16. 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.[1]

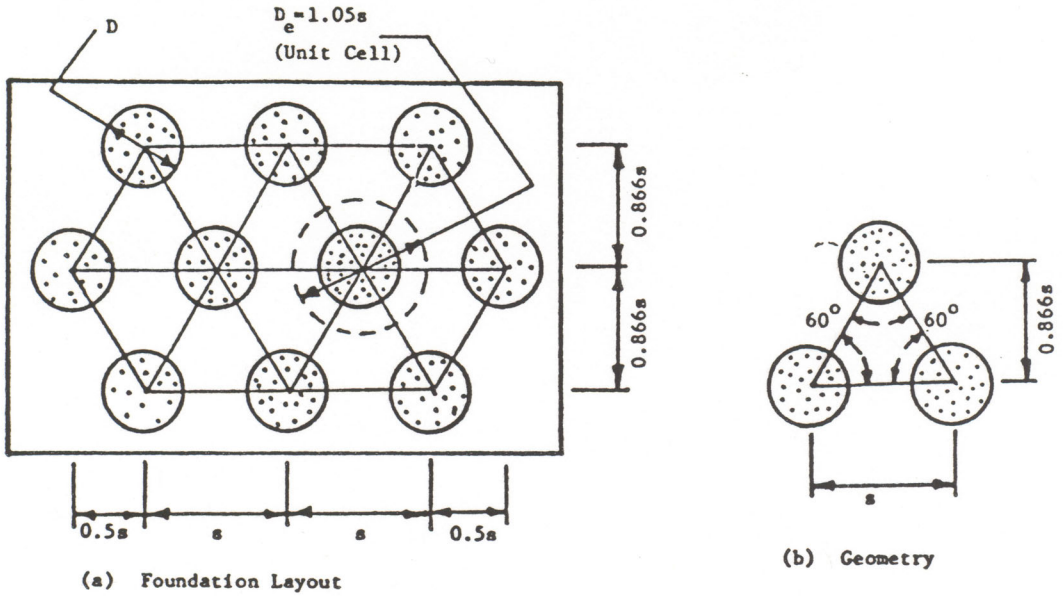


Figure 15. Equilateral Triangular Pattern of Stone Columns.

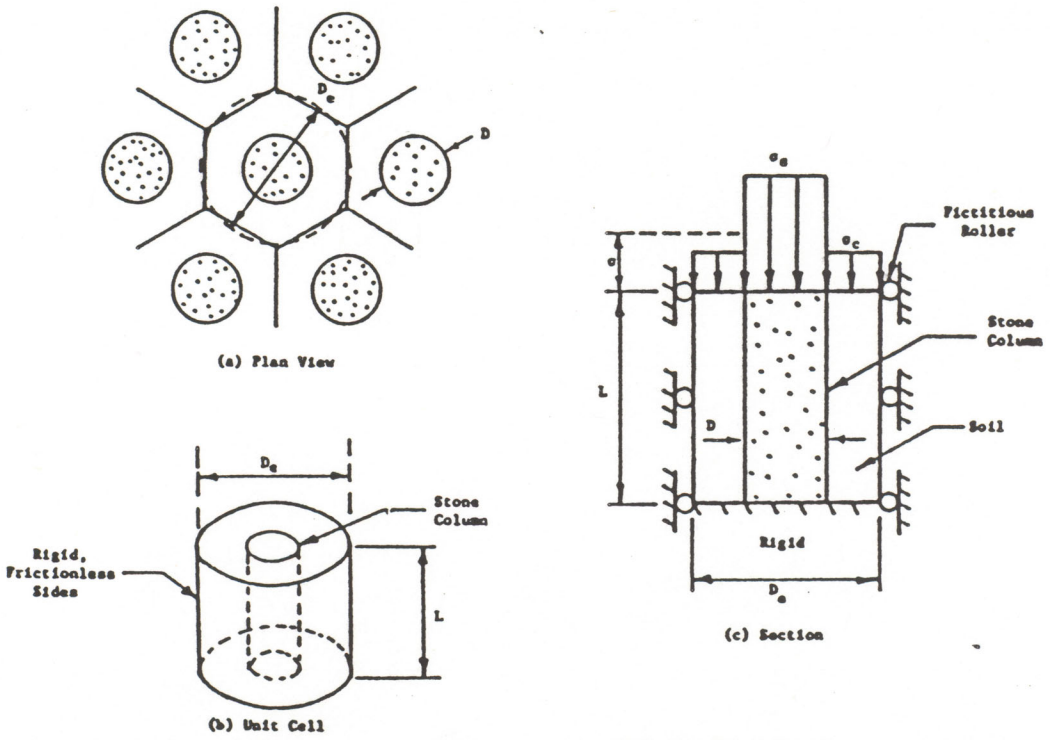


Figure 16. Unit Cell Idealization.

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_s, is defined as the fraction of soil tributary to the stone column replaced by the stone:

$$\alpha_s = \frac{A_s}{A} \qquad (1)$$

where "A_S" is the area of the stone column after compaction and "A" is the total area within the unit cell.[(1)]

Typical ratios used are in the range of 0.10 to 0.40.

The literature also describes the ratio a_s, the *area improvement ratio,* which is the inverse of an area replacement ratio.

Spacing and Diameter

Stone column diameters vary between 0.45-1.2 m (1.5-4 ft.), but are typically in the range of 0.9-1.1 m (3.0-3.6 ft.) for the dry method, and somewhat larger for the wet method.

Triangular, square, or rectangular grid patterns are used with center-to-center column spacing of 1.5-3.5 m (5-12 ft.). For footing support, they are installed in rows or clusters. For both footing or wide area support, they may extend *beyond the loaded area*.

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 (stress in stone column divided by stress in in-situ soil) 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 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.[1,16]

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 capacity calculations.

Once a stress ratio has been assumed or determined, the stress on the stone column, σ_s, and on the surrounding soil, σ_c, can be calculated for each replacement ratio, α_s, and any average stress condition, q, that would exist over the unit cell as follows:

$$n = \frac{\sigma_s}{\sigma_c} \tag{2}$$

For equilibrium of vertical forces for a given a_s

$$q = \sigma_s(\alpha_s) + \sigma_c(1 - \alpha_s) \tag{3}$$

For a given stress concentration factor, the stress on the unimproved soil is:

$$\sigma_c = \frac{q}{[1 + (n-1)a_s]} \tag{4}$$

and on the stone column:

$$\sigma_s = \frac{nq}{[1 + (n+1)a_s]} \tag{5}$$

Time Rate of Settlement

Stone columns act similarly to wick drains in decreasing the distance in which water has to flow in the radial direction for primary consolidation to occur.

Methods outlined in the companion technical summary on "Wick Drains" should be used to compute the time rate of settlement.

Bearing Capacity

In determining the ultimate bearing capacity of a stone column or a stone column group, the possible modes of failure to be considered are illustrated in Figures 17, 18, and 19. Caution should be given to avoiding local bulging failures due to very weak or organic layers of limited thickness (Figure 18). 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.

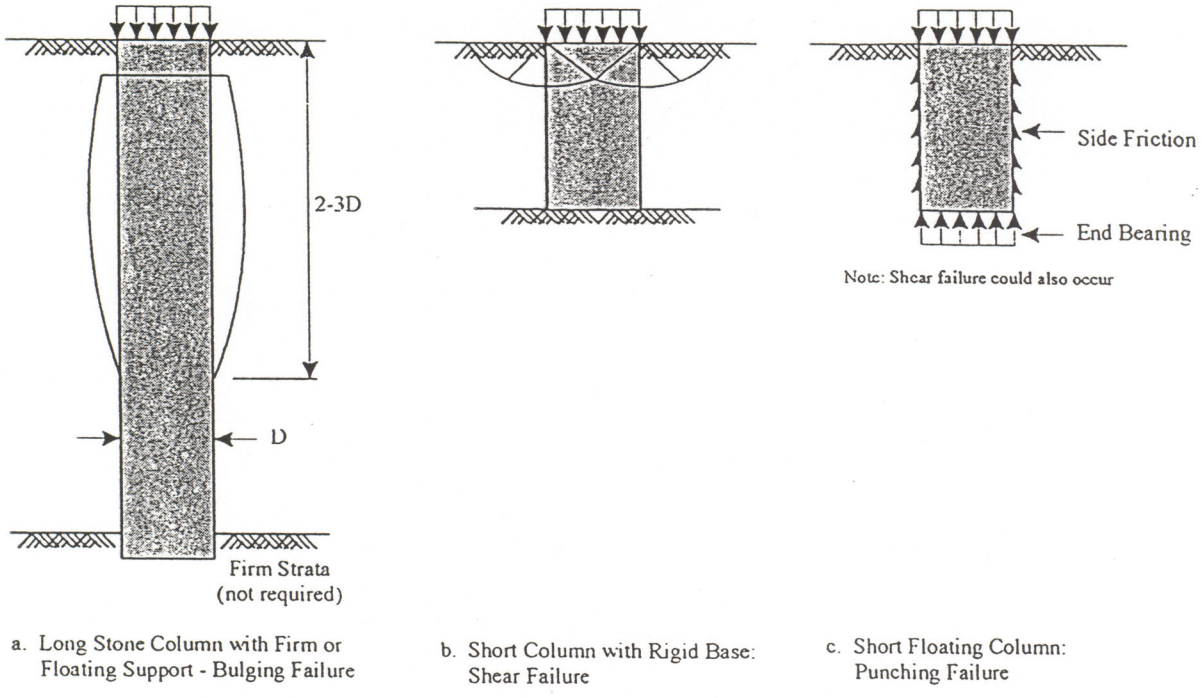


Figure 17. Failure Modes of a Single Stone Column in a Homogenous Soft Layer.

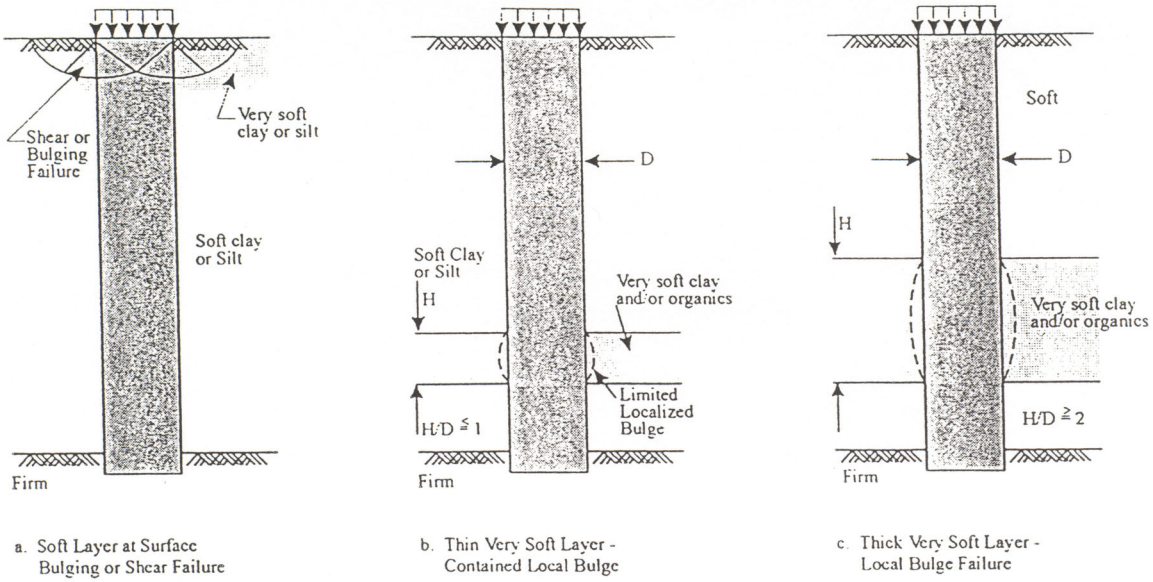


Figure 18. Failure Modes of a Single Stone Column in a Non-Homogeneous Cohesive Soil.

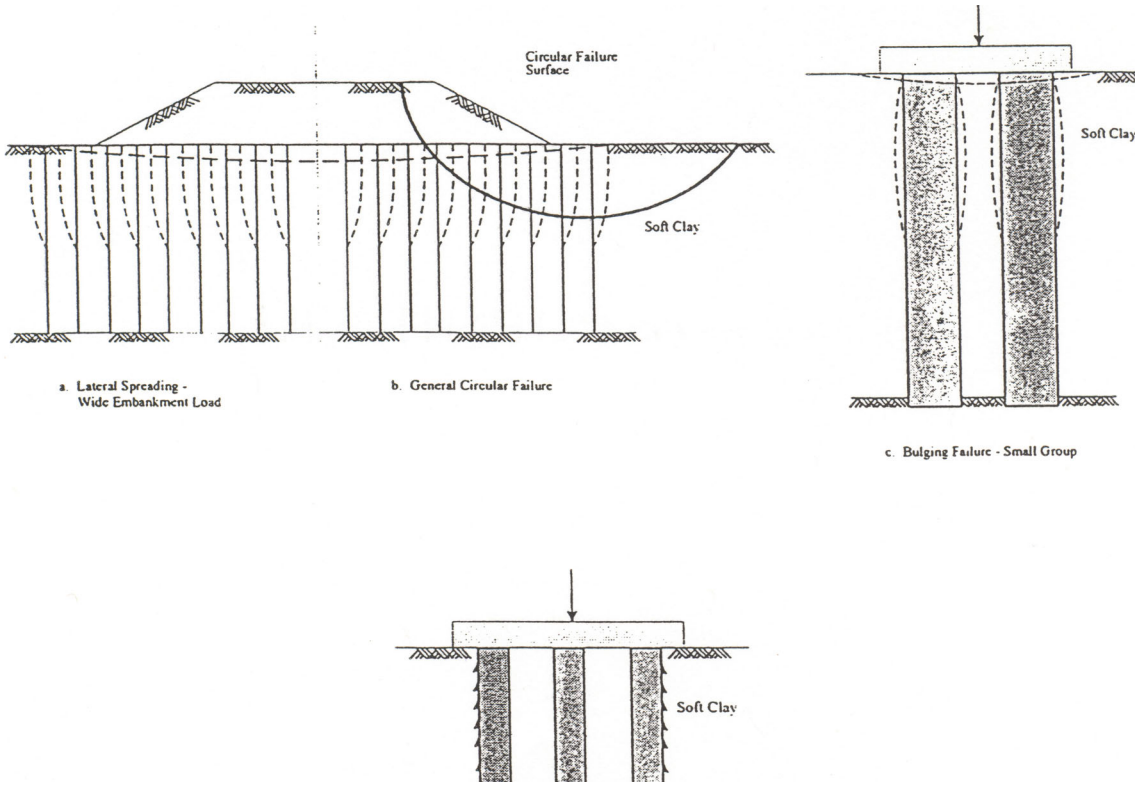


Figure 19. Failure Modes of Stone Column Groups.

The rational prediction of the bearing capacity of stone column groups loaded by either a rigid foundation or a flexible load, 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 capacity of an isolated stone column or a stone column located within a group can be expressed in terms of an ultimate stress applied over the stone column:

$$q_{ult} = cN_c \qquad (6)$$

where q_{ult}, c, and N_c are the ultimate stress that the stone column can carry, the undrained shear strength of the surrounding cohesive soil, and the bearing capacity factor for the stone column ($18 \leq N_c \leq 22$), respectively.

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 factor of safety of 3 is recommended for design if using Equation 6.

Typical single column design loads of 180-240 kN (20-30 tons) can be used in soft to medium stiff clays.

For rammed aggregate piers, the use of higher column loads is reported.[13]

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.[1] 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 (a_s) or area improvement ($1/a_s$) 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 on Figure 20.[24]

It should be noted in Figure 20 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_c) to the in-situ soil modulus (E_s), 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 *"Design and Construction of Stone Columns"* is roughly equivalent to the Balaam relationships shown on Figure 20 and represents an average or lower bound estimate suitable for preliminary analyses. For settlement analyses, a stress ratio of 3 is often used to determine the settlement of the in-situ soil between columns.

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. 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 50-80 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.

Shear Strength Increase

For slope stability analyses, an average shear strength of the soil/stone column composite material is used along the sliding surface, as shown on Figure 21.[18]

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.[24]

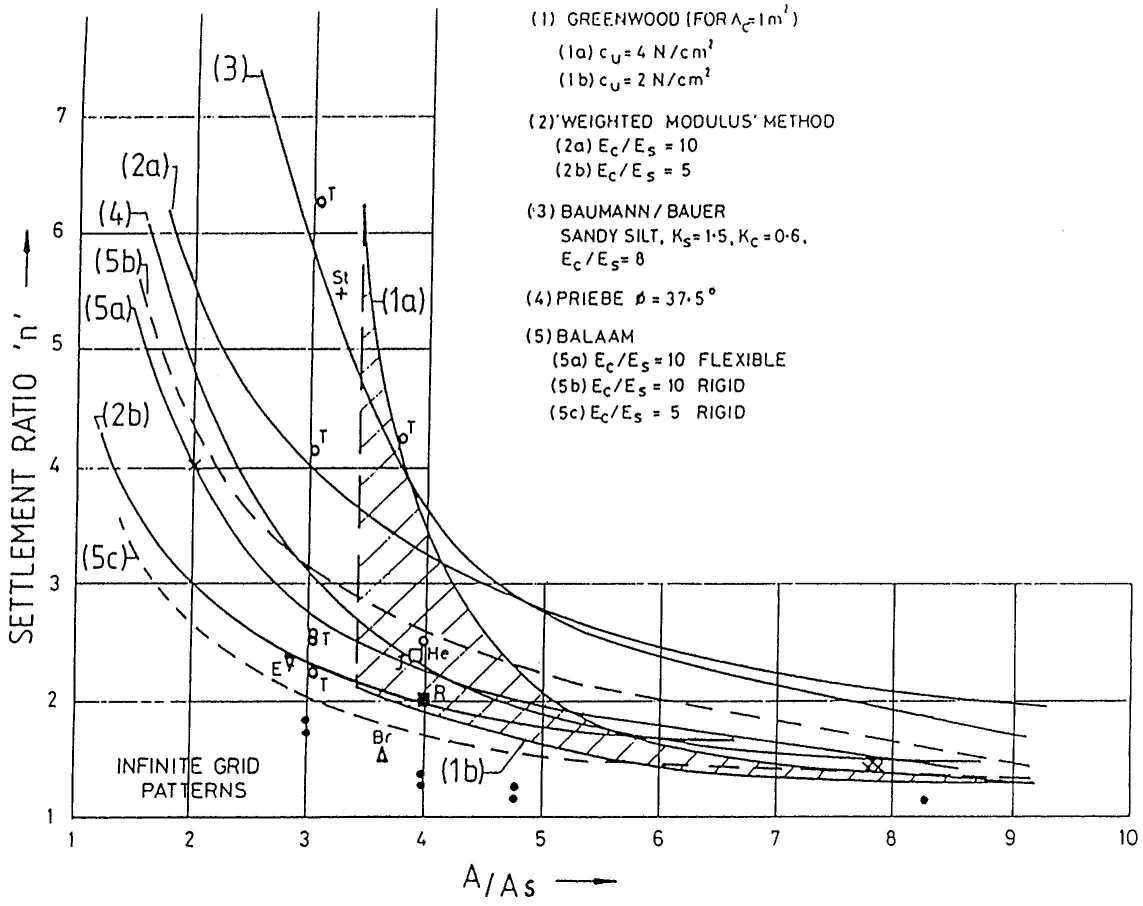


Figure 20. Comparison of Elastic Theories and Field Observations.[22]

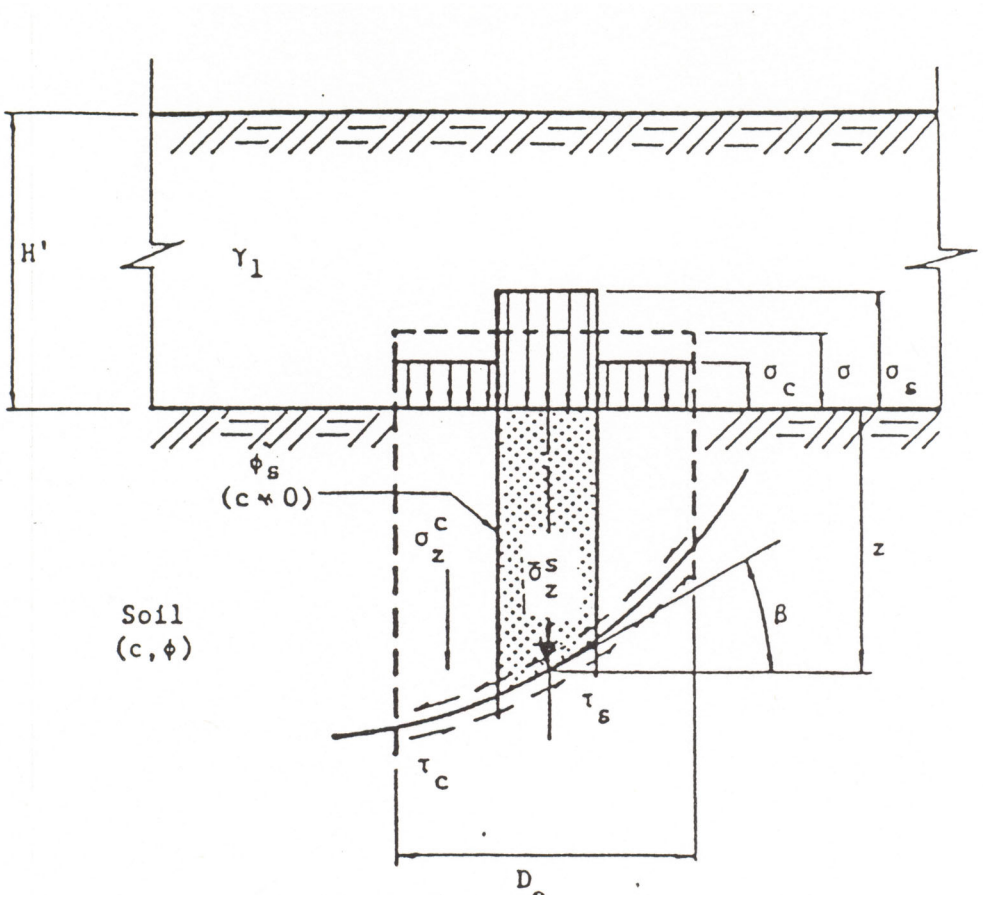


Figure 21. Notation Used in Average Stress Method Stability Analysis.

The average strength and average unit weight parameters ($\tau; \bar{\gamma}$) can be determined as follows:[1]

$$\tau = (1 - a_s)\, c_u + a_s\, \sigma_v \tan \phi_s \quad (7)$$

$$\bar{\gamma} = \gamma_c\, a_s + \gamma_s\, (1 - a_s) \quad (8)$$

where τ = average weighted shear strength

c_u = undrained shear strength of in-situ soil

$\bar{\gamma}$ = average unit weight

γ_s, γ_c = unit weight of soil and stone column

ϕ_s = angle of friction for stone column

σ_v = stress due to embankment loading

a_s = area replacement ratio

For design, the angle of internal friction ϕ_s of the stone column typically used varies from 40-45 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 $N_c = 0$ 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.

Other computational methods such as the lumped moment method, have been successfully used.[1] For a complete description of the stability analysis methods refer to FHWA *"Design and Construction of Stone Columns"*.[1]

4.3 SEISMIC DESIGN OF STONE COLUMNS

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",*[12] *Soil Improvement for Earthquake Hazard Mitigation,*[4] *Ground Improvement Ground Reinforcement, Ground Treatment: Development 1987-1997,*[3] *Advances in the Design of Vibro Systems for Improvement of Liquefaction Resistance*[30] provide recommendations on how to quantify the benefits of ground improvement 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.

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)[31] developed empirical liquefaction curves which correlate cyclic stress ratio to corrected penetration resistance. The cyclic stress ratio (CSR_{EQ}) is determined as follows:

$$CSR_{EQ} = 0.65\ (a_{max}/g)\ r_d\ (\sigma_v/\sigma_v') \qquad (9)$$

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[32]:

r_d	= 1.0-0.00765z	for $z \leq$ 9.2 m (30 ft.)
r_d	= 1.174-0.0267z	for 9.2 m (30 ft.) $< z \leq$ 23 m (75 ft.)
r_d	= 0.744-0.008z	for 23 m (75 ft.) $< z \leq$ 30 m (100 ft.)
r_d	= 0.5	for z < 3 0 m (100 ft.)

Figure 22 shows the relationship between stress ratio causing liquefaction and corrected SPT "N" values for silty sands for a magnitude 7.5 earthquake. This Figure may be used to estimate the required improvement in soil density to prevent liquefaction.

Spacing

The spacing of the stone columns may be determined using Figure 22 to determine the corrected SPT "N" value. Table 3 from the Vibro-Compaction Technical Summary may be used to correlate SPT "N" values to relative density. Figure 7a (from the Vibro-Compaction

Technical Summary) is then used to estimate the required stone column spacing to improve the soil to the required penetration resistance at the mid-point between columns.

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.[25] This recommendation can be achieved by selection of the gradation for the stone column, with due regard to piping considerations outlined below.

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:

$$20D_{S15} < D_{G15} < 9D_{S85} \quad (8)$$

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.

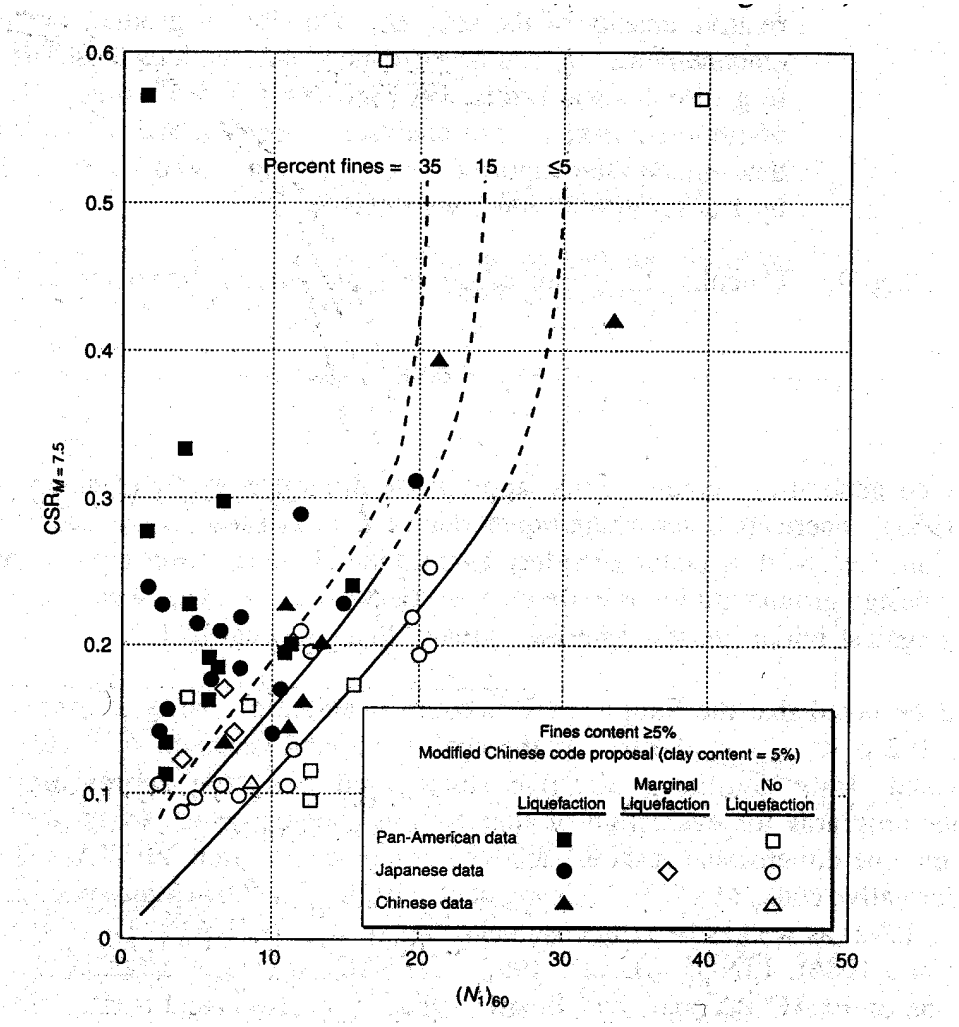


Figure 22. Relationship between Stress Ratio Causing Liquefaction and $(N_1)_{60}$ Values for Sands for 7.5 Magnitude Earthquakes (Seed et.al 1985).

4.4 VIBRO-CONCRETE COLUMNS (VCC)

The VCC technique combines the ground improvement advantages of vibro systems with the load carrying advantages of piles.

The design of the VCC improved site is essentially the same as that performed for a driven or bored pile foundation, except that improved soil parameters are used. For large area loads, the VCC system is typically overlain by a granular mat and sometimes reinforced by a geosynthetic to distribute the applied loads evenly to the VCC system (see Technical Summary on Column Supported Embankments). At the ground surface, a slight mushrooming of the concrete column occurs that also assists with the load transfer.

Unlike a deep pile foundation system, however, the vibro-concrete columns are not connected to the structure. Therefore, the planned structure can be designed with a standard shallow foundation system or, in the case of embankments, with a uniform bearing pressure. The addition of the granular mat is not needed if a sufficient thickness of surface granular soil is present, which will be densified as a result of the VCC construction and act to distribute the loads to the VCC system.

In general, the vibro-concrete columns are used without reinforcement. However, if required, single bars or short cages (< 6 m (20 ft.)) can be placed into the completed columns.

4.5 GEOPIER RAMMED AGGREGATE PIERS

The design concept used for Geopier 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 conventional stone columns, resulting in a higher design stress ratio than previously identified for stone columns. Although the Geopier design manual suggests stress ratios of 10 or higher, a stress ratio of 4 to 6 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 (up to 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. A minimum replacement ratio of 0.33 is recommended.[13]

For details, refer to *"Geopier Foundation and Soil Reinforcement Manual"*.[13]

4.6 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.[26] A program should be specified, regardless of the contracting method.

A combination of load tests on stone 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 capacity, (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 stone-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 20-25 percent of ultimate should be used. A final acceptance criterion of 25 mm (1 in.) of settlement at 150-200 percent of the allowable/design load appears to be a reasonable criterion.

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 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 150-200 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 electric 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 Verification Test" have been developed as quality control checks. For details, consult the Geopier Design Manual.[13]

CHAPTER 5
CONTRACTING METHODS AND SPECIFICATIONS

Like other methods of specialty construction, unless the specifying agency has expertise in the design, construction, and inspection of stone 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 stone column technique, a method specification may be utilized.

As indicated, the wet method of construction and the dry, bottom feed method are both used. Both a method specification and a performance specification are presented for the wet, top feed method, and a performance specification is presented for the dry, bottom feed method.

5.1 METHOD SPECIFICATION

The following method specification was originally published in *"Design and Construction of Stone Columns"*[1] and has been updated to reflect current practice.

NOTE: ***This specification should not be used directly, but should be adapted to the specific conditions and needs of the project.***

Guide Specification
Wet Top-Feed Stone Columns

General

Ground improvement should be performed by constructing stone columns formed by deep vibratory techniques using select granular material. The principal items of work included in this specification are:

1. Construction of stone columns, complete in-place, including layout.

2. Furnishing select granular material, as required for stone columns.

3. Furnishing equipment, electrical power, water, and any other necessary items for stone column installation.

4. Control and disposal of water resulting from stone column construction operations.

5. Load testing of stone columns, as specified.

The installation of all stone columns under the contract should be the responsibility of one specialty contractor. No part of the contract may be sub-let without prior approval of the engineer. The contractor should furnish all supervision, labor, equipment, materials, and related engineering services necessary to perform all subsurface ground improvement work.

Requirements of Regulatory Agencies

Prevention of Nuisance

The contractor should comply with all laws, ordinances, and other regulatory requirements governing the work, including those pertaining to the prevention of nuisance to the public and adjoining property owners by noise, impact, vibration, dust, dirt, water, and other causes. The contractor should immediately discontinue any construction or transportation method that creates any such nuisance and perform the work by suitable lawful methods at no extra cost to the owner.

Requirements of the Contractor

Disposal of Water

The contractor should (1) meet all applicable laws and regulations concerning surface runoff, siltation, pollution, and general disposal of the effluent from the construction of the stone columns and general site work; (2) construct and relocate temporary ditches, swales, banks, dams, and similar facilities, as necessary to control the flow of surface water during the work; (3) construct silt settling ponds, as required in locations indicated or approved. Ensure that earth banks and water control devices are safely designed, and prevent inadvertent discharge into watercourses off the site. Stockpile and dispose of all silt as approved by the engineer; and (4) remove settling ponds and other structures when no longer required, and re-grade the areas for acceptable drainage, as specified for site grading.

Materials

The contractor should notify the engineer in writing of proposed sources for select granular material at least 14 days before importation operations begin. This material will be sampled at the source and tested by the owner/engineer to determine compliance with the requirements specified.

Select Granular Material

The crushed stone for column backfill should be clean, hard, unweathered stone free from organics, trash, or other deleterious materials. When subjected to the magnesium sulfate soundness test (ASTM C88), the percent weight loss should be no more than 15 percent. When tested according to ASTM C131, the crushed stone should have a maximum loss of 40 percent at 500 revolutions. The gradation should conform to the following for the vibro-replacement process:

Table 1. Gradation for various sieve sizes.

Sieve Size (inches)	Alternate No. 1 Percent Passing	Alternate No. 2 Percent Passing	Alternate No. 3 Percent Passing	Alternate No. 4 Percent Passing
4.0	--	--	100	--
3.5	--	--	90-100	--
3.0	90-100	--	--	--
2.5	--	--	25-100	100
2.0	40-90	100	--	65-100
1.5	--	--	0-60	--
1.0	--	2	--	20-100
0.75	0-10	--	0-10	10-55
0.50	0-5	--	0-5	0-5

The contractor will be required to submit representative test results and the materials will be accepted based on certification. Contractor submitted certification and representative test results required are as follows:

1. Gradation in accordance with AASHTO T-27.

2. Specific Gravity in accordance with ASTM C127.

3. Density of loose select material in accordance with ASTM C29.

4. Density of compacted select material in accordance with ASTM C29.

A series of tests shall be performed for each 5,000 tons of stone or sand furnished from each source. Additional tests may be required by the engineer, and failing tests will be paid for by the contractor.

Water

Fresh, brackish, seawater, or any combination, free of all substances deleterious to the work, may be used.

Equipment and Methods

At the beginning of the project, stone columns should be installed at locations shown on the plans or designated by the engineer for the purpose of establishing quality control procedures.

Vibrator

Stone columns should be installed by jetting, using vibratory probes 350-480 mm (13.8-19 in.) in diameter (not including the fins). The vibrator should have an eccentric mass located in the lower part of the probe that should be capable of developing the required vibration characteristics at a frequency of 1600-3000 rpm. The vibrator should be driven by a motor having at least a 100 kW rating that is capable of developing a minimum centrifugal force, in starting, of 150 kN (33.8 kips). The minimum double amplitude (peak-to-peak measurement) of the probe tip should be not less than 14 mm (0.55 in.) in the horizontal direction when the probe is in a free suspended position. Typical frequency of vibration is 1800 rpm.

Installation

The construction technique and probe should be capable of producing and/or complying with the following:

1. The probe and follower tubes should be of sufficient length to reach the elevations shown on the plans. The probe, used in combination with the flow rate and available pressure to the tip jet, should be capable of penetrating to the required tip elevation. Preboring of stiff lenses, layers of strata is permitted.

2. The probe should have visible external markings at 0.33-m increments to enable measurement of penetration and repenetration depths.

3. Provide for supplying sufficient quantity of water to maintain an open hole to allow adequate space for stone backfill placement around the probe. The flow of water from the bottom jet should be maintained at all times during backfilling to prevent caving or collapsing of the hole and to form a clean stone column. An average flow of 11-15 m^3/hr of water should be maintained throughout construction. The flow rate will generally be greater during penetration, and will decrease as the stone column is constructed.

4. After full penetration, the vibrator should be partially removed twice to flush out the hole. The probe should not, however, be completely removed from the hole.

5. Form the column by adding stone to fill the hole in 0.6-to 1.2-m lifts. Compact the stone in each lift by repenetrating it with the probe so as to densify and force the stone radially into the surrounding in-situ soil until the amperage required to drive the electric motor reaches a predetermined level.

6. The previous stone placement and stone compaction procedures should be consistently repeated to ensure that each column is continuous through its length.

During construction, if the stone columns are consistently over or under the average effective diameter of ___ meters, as defined in the tolerances section, and the workmanship and material have been consistent with those used in previously acceptable work, this may indicate that the soil conditions have changed from those encountered during the earlier work. The contractor should cease operations in the immediate area of work and notify the engineer. The engineer will make a determination of whether it is necessary and the extent to which it is necessary, to adjust the pattern and spacing.

Workmanship

Determination of the quality and adequacy of workmanship employed in the installation of the stone columns in the work will include consideration of the contractor's consistent use of procedures, methods, and performance rates as used in installing the initially acceptable stone columns.

Tolerances

Location

No stone column should be more than 100 mm (3.9 in.) from the center location at the working platform level, as shown on the approved plans, except as specified in the obstructions section. The axis of the stone column should not be inclined from the vertical by more than 50 mm in 3 m (2 in. in 10 ft.).

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. If the columns do not meet the above requirements, then the installation operation must be adjusted to produce the specified diameters.

The average effective stone column diameter should be calculated using the in-place density of the stone and the weight of stone used to fill the hole. The weight of stone required to construct the stone column should be based on the equivalent number of full buckets dumped down the hole and the loose stone density as specified in the materials section.

Obstructions

A 380-mm (15 in.) maximum horizontal deviation from indicated column location will be allowed without prior authorization from the engineer when an obstruction is encountered; the presence of any obstruction should be reported to the engineer and described in the records. When a deviation greater than 380 mm (15 in.) is caused by an obstruction, the contractor should stop work, move to another location and immediately notify the engineer. The engineer may at his option authorize one or several of the following: (1) position the stone column point a short distance away from the original position, (2) additional stone columns to bridge the obstruction, (3) remove the obstruction, replace removed soils, and jet the column hole in the indicated location or (4) perform other removal or relocated operations. The owner will pay the contractor for authorized work to remove obstructions or for performing directed relocation operations, except shifting the compaction point, based on accepted contract unit prices.

Construction Records

The contractor shall provide competent and qualified personnel to continuously observe the installation and furnish to the engineer recorded logs of the following data to be obtained during column installation:

- Stone column reference number.
- Elevation of top and bottom of each stone column.
- Number of buckets of stone backfill in each stone column.
- Amperage achieved with 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.

(The owner should furnish a full-time inspector to observe stone column construction.)

The records of the above information signed by the contractor's representative and the owner's inspector shall be submitted to the engineer each day.

The stationing, top elevation, limits, pattern, spacing, and approximate depths for the stone column work are shown on the plans. The contractor shall prepare construction drawings showing specific stone column location, identification number, and estimated depth of each column. These drawings are to be submitted to the engineer for approval in accordance with contract requirements. During the progress of work these drawings shall be annotated to show the stone columns completed each day.

At the end of the work, a report shall be prepared by the contractor and submitted to the owner giving details of the plant and methods used, production rates, and the performance of the site during treatment, together with all load test results and calculations based on the data obtained during the stone column construction.

Method of Measurement

The accepted quantity of stone columns, including test columns, shall be measured in total linear meters of all columns complete in-place. Measurement will be from the bottom of each column to the elevation given on the contract plans. Measurement of each column will be to the nearest 0.5-m (1.6 ft.).

Basis of Payment

The contractor shall be paid a lump sum amount for the set-up and removal of facilities and equipment for stone column production. In addition, stone columns shall be paid for at the contract unit price per linear meter. The above payments will constitute full compensation for providing records, logs, and reports, and for providing all labor, supervision, tools, equipment, materials, and incidentals necessary to complete the items specified by the engineer. (Note: The working platform material and placement is a separate pay item.)

Short-Term Load Tests

(Load tests may not be required on all stone column projects.)

The contractor should furnish all required reaction beams, hydraulic jacks, gauges, and instrumentation for the tests. The test method should be in accordance with ASTM D-1143 Quick Load Test Method.

1. *Equipment*

    a. Capacity of Load Test Equipment. The test equipment should be capable of safe application of two times the specified design load for preliminary tests for non-production individual stone columns, and one and a half times the specified design load in the case of production stone columns.

    b. Reaction Piles

        (1) Deadweight. If deadweight is used to provide the reaction for the hydraulic jack, it should be supported on a suitable platform to allow safe access to the loading and measuring equipment at all times. The nearest edge of the platform supports should be at least 2.5 m (8 ft.) from the periphery of the stone column.

        (2) Reaction Anchors. If tension anchors are used to provide the reaction for the hydraulic jack, these anchors should not be greater than 2.5 m (8.2 ft.) from the stone column periphery.

    c. Load Measurement. The test load should be applied vertically and concentrically to the stone column by means of a calibrated hydraulic jack meeting the test requirements. The applied load should be measured by an

approved load cell and calibrated pressure gauge having divisions not exceeding 2 percent of the maximum load to be applied. A certificate of calibration for the load cell or gauge, obtained within three months prior to the test, should be provided.

d. Deflection Measurement. Observations of vertical deflection of the head of the stone column should be made with a minimum number of two dial gages having 50-mm (2-in.) travel and graduated in 0.025-mm (0.001-in.) divisions. The tips of the stems of the dial gages should rest on metal plates securely bedded on the head of the steel loading base.

   The readings should be referenced to two rigid steel beams, the ends of which should be fixed to reliable steel supports. The supports should be located not closer than 2.5 m (8 ft.) from the center of the test stone column, and away from the influence of the reaction system.

e. Protection of Measuring Equipment. The measuring equipment should be protected throughout the period of the test against the adverse effects of rain, sun, frost, vibration, and other disturbances that may affect its reliability.

Application of Load

The loads should be applied in increments in accordance with ASTM D-1143.

Each load increment should be maintained for 15 minutes before the next increment is added. Unloading should then take place in five equal decrements, with each intermediate load being maintained for a minimum period of 15 minutes.

Zero load, at the end of the cycle of unloading, should be maintained for a minimum of one hour and/or until the rate of recovery does not exceed 0.15 mm (0.006 in.) per hour.

Notification, Supervision, Reports

The period between the construction of a stone column and the commencement of the application of the test loads should be at least 24 hours. The contractor should give at least 48 hours notice of the commencement of each load test to the engineer.

The contractor should keep the test under continuous supervision to the satisfaction of the engineer. All necessary facilities should be provided to enable the engineer to verify

readings during the progress of the test. The contractor should transmit to the engineer, within one week of the test completion, four copies of all load test records and results in graphical form. This information should include a load deflection curve plotted to scale so as to approximately fill a standard sized page.

5.2 PERFORMANCE SPECIFICATION

The following is a sample performance specification for wet, top-feed stone column construction.

NOTE: ***This specification should not be used directly, but should be adapted to the specific conditions and needs of the project.***

Within the parentheses of this sample specification are a series of terms or descriptions that will allow the designer the flexibility to adapt to the specific requirements of the project.

Guide Specification
Stone Columns by Wet Feed

Introduction

Vibro-Replacement is a technique utilizing specially developed vibrators to direct compactive energy at the required improvement depth. This compactive energy results from electrically or hydraulically driven eccentric weight assemblies to apply a radial energy in the horizontal plane to surrounding soil materials. Improvement by stone column installation is achieved by increasing the in-situ density and shear strength, and reinforcing and locally replacing the subsurface materials with a select granular backfill.

Documentation

The contractor shall furnish shop drawings to the engineer for review 2 weeks prior to the start of work, indicating the spacing, location, and depth of the vibro points to achieve the criteria outlined in this specification.

A daily log shall be submitted to the engineer by the contractor to include: recording of probe number, start/finish time of probe, depth of treatment, approximate backfill quantities, and indication of relative ammeter increases.

Any change in the predetermined vibro program necessitated by a change in the subsurface conditions should be immediately reported and submitted to the engineer.

Scope of Work

This specification details the technical and quality assurance requirements for furnishing all supervision, labor, material, equipment, and related services necessary to perform all soil improvement by stone column installation as specified.

The work includes subsurface soil improvement and delivery and placement of all stone backfill necessary in the improvement process. Soil improvement by this method will be limited to the areas shown on the contract plans for *(mat/column footings, wall footings, dams, embankment slopes)*.

Applicable Documents

All Vibro-Replacement work shall be based upon the following: foundation plans, geotechnical reports, design criteria, etc. The following documents are referenced elsewhere in this specification:

ASTM D1586, Penetration Test and Split Barrel Sampling of Soils

ASTM D 3441, Deep, Quasi-Static, Cone and Friction-Cone Penetration Tests of Soil

ASTM D 1143, Piles Under Static Axial Compressive Loading

Qualifications

The contractor performing the Vibro-Replacement shall have a minimum 3-year experience, documenting 5 recent, successful projects completed. References asserting this documentation should be provided to the engineer a minimum of 30 days prior to construction.

Requirements

Site Preparation

The contractor shall provide a firm working platform on which heavy equipment can be operated safely without platform maintenance, aside from water diversion and containment, where required.

The contractor shall accurately locate building corners/grid, lines/footing, locations/ utilities, etc.

The contractor shall provide access and maintenance thereof for the stone column equipment, work force, and delivery of materials to the work site.

Backfill Materials

Backfill material to be used in the Vibro-Replacement work shall consist of hard, durable aggregate with a gradation generally between 7.5-75 mm (0.30-3 in.).

Equipment and Procedure

Specific equipment and installation procedures to achieve the specified criteria are the contractor's responsibility. However, the following general guidelines are identified:

The contractor should use a down-hole vibrator capable of providing at least 60 kW of power and 15 tons of force.

After penetration to the treatment depth, the vibrator should be slowly retrieved in 300-500 mm (12-20 in.) increments to allow backfill placement.

The vibrator should repenetrate each new increment to obtain an observed amperage increase.

(Amperage increase and backfill quantities are contingent on the type of vibrator, type of backfill, in-situ soil conditions, and contractor's procedure.)

Acceptance Criteria

Any of the following acceptance criteria may be specified.

Cohesive Soils

The performance criteria could require the stone columns to provide any combination of the following: 1) an average allowable axial capacity, typically 180-270 kN (40-60 kips) per column in soft to medium stiff clays; 2) an average bearing pressure, typically 150-200 kN/m^2 (3000-4000 psf); 3) a total settlement that is less than a specified value, typically 25-

100 mm (1-4 in.); 4) a limiting differential settlement, a typical angular distortion for soft clays is 1/300 to 1/500; and/or 5) a minimum factor of safety, usually 1.5, against slope instability.[26]

Granular Soils

Soil improvement of the underlying, granular soils as necessary to provide a relative density of ___%; an average corrected 'N' value for SPT testing; an average cone value for CPT testing and sufficient improvement in the slightly cohesive soils to provide an average modulus value and a bearing capacity of ___ and limitation of total/differential settlement to 1/300 to 1/500 as for clays.

Restrictions

The contractor shall be responsible for obtaining all State and municipal permits and conforming to all State and local regulations.

(Environmental and utility restrictions should be listed within this section.)

Quality Assurance

Testing and Inspection

All testing to determine specification compliance shall be provided by an *(independent testing agency or the owner)* and may consist of either standard penetration testing (SPT-ASTM D 1586); static cone penetration testing (CPT-ASTM D 3441); or plate load tests (ASTM D 1194). The same test method should be utilized both before and after the soil improvement work.

The engineer will inspect the contractor to ensure performance of the Vibro-Replacement work. This inspection may include any or all of the following: observance of the contractor's procedures, recording of backfill quantities, and recording of ammeter information.

Measurement and Payment

Lump sum for all materials and labor to achieve specified criteria. *Performance specifications should delineate the area of densification/reinforcement, as well as the criteria to be achieved. On this basis, the experienced stone column contractor can evaluate the soil*

and project conditions to provide the most economic lump sum price. Unit area or volume rates may be requested on an area or load basis to add/deduct quantities.

Mobilization/demobilization and testing are typically separate lump sum figures.

5.3 DRY FEED PERFORMANCE SPECIFICATION

NOTE: ***This specification should not be used directly, but should be adapted to the specific conditions and needs of the project.***

Guide Specifications
Stone Columns by Dry Feed

1.1 Description

This work is a performance program developed by the Contractor to meet the design foundation bearing and settlement criteria, and consists of designing and furnishing the material for, and placement of, the stone columns using dry, bottom-feed installation techniques in locations and to depths as shown on the project documents and the approved shop drawings.

[Purpose of the stone column program should be provided here, as well as acceptance criteria.]

1.2 References

A. A geotechnical report prepared by *[the geotechnical engineer]*.

B. Standard Specifications, as appropriate.

C. Appropriate ASTM and AASHTO standards for acceptance tests.

1.3 Submittals

A. The Contractor shall submit three copies of the following documents to the engineer a minimum of 1 month prior to the installation of the stone columns.

1. Evidence of successful installation of stone columns on five or more projects under similar conditions using the dry bottom feed installation technique within the last 3 years. The documentation to be submitted includes references for the specific projects. The references consist of the Owner of the project and the project Engineer, including his name, address, and telephone number.

2. Construction drawings showing stone column locations, depths, and identification numbers.

3. A description of the equipment and construction procedures to be used.

4. Certification that the project Superintendent possesses a minimum of 3 years of method-specific experience.

5. The source of the proposed stone column backfill material and the gradation and band widths the Contractor proposes to use. Upon approval of the backfill source and gradation, the Contractor shall maintain this gradation throughout the stone column installation.

B. The initial stone columns placed to the required depths and locations as shown on the Contractor's plans, and meeting the requirements of this specification serves as the quality control basis for the remainder of the stone columns to be installed upon approval by the Engineer. Any deviations from the method utilized in the initial column placement in the subsequent columns must be approved by the Engineer before they are applied to production columns. Perform column load tests in accordance with quick load test standards of ASTM D-1143 at locations selected by the Engineer.

C. During construction, the Contractor shall submit three copies of daily progress reports in writing to the Engineer that detail the following:

- Stone column identified by location number.
- Date constructed.
- Elevation of top and bottom of each stone column.
- Estimate of ground heave or subsidence.
- Vibrator power consumption during penetration and compaction of each increment of stone column constructed.
- Jetting pressure (air).

- Details of obstructions, delays and any unusual ground conditions.
- Quantity of stone placed in each 1.5-meter interval of column and total aggregate used per stone column.

D. At the completion of the stone column work, the Contractor shall submit a report to the Engineer detailing the plant and methods used, production rates, the performance of the site during treatment, and that the site meets the criteria established for this project.

1.4 Basis of Acceptance

The basis of acceptance for the stone columns shall be visual inspection by the Engineer, who will consider results of all acceptance tests, as well as consistent use of procedures, methods, and construction performance rates for the columns in the various treated areas, compared to those used in installing the initially accepted stone columns.

(A lump sum basis of payment is typically used with a performance specification. The lump sum basis of payment allows the contractor to select the most efficient method of stone column construction to satisfy the performance criteria. However, the area and depths of improvement and the performance criteria must be clearly defined if a lump sum price is used. The lump sum should provide full compensation for furnishing all labor, including a qualified supervisor, materials, tools, supplies, equipment, and incidentals necessary to design, install, and proof test the stone columns constructed during the production phase of the construction. The effluent handling and disposal, and the initial load testing can be covered in a lump sum price or as a separate pay item, depending on the project.)

2.1 Materials

A. The aggregate used for the construction of the stone columns shall meet the following requirements:

(*Gradation and soundness requirement should be provided here.)*

3.1 Construction Requirements

A. The following are minimum requirements:

1. Install the stone columns with a down-hole vibrator capable of providing a minimum of 100 kW and a minimum centrifugal force of 150 kN (34 kips) gyrating about a vertical axis. The minimum double amplitude (peak-to-peak measurement) of the probe tip shall be not less than 14 mm (0.5 in.) in a horizontal direction when the probe is in a free suspended position. Limit the air pressure used to a range between 200-400 kPa (30-60 psi) to prevent excessive ground heave from air pumped in the ground.

    a. The probe and follower tubes shall be of sufficient length to reach the elevations shown on the plans. The probe, used in conjunction with the available pressure to the tip jet, shall be capable of penetrating to the required tip elevation. Preboring of silt lenses, layers, or strata, if encountered, is permitted.

    b. The probe and follower tubes shall have visible markings at 0.3-m (1 ft.) increments to enable measurement of penetration and repenetration depths.

    c. Provide methods for supplying to the tip of the probe a sufficient quantity of air to widen the probe hole to allow adequate space for stone backfill placement around the probe. Maintain the flow of air from the bottom jet during backfilling to prevent caving or collapsing of the hole and to form a clean stone column.

    d. After penetration to the treatment depth, slowly retrieve the vibrator in 300-500 mm (12-20 in.) increments to allow backfill placement.

    e. Compact the backfill in each lift by repenetrating it at least twice with the horizontally vibrating probe so as to densify and force the stone radially into the surrounding in-situ soil. Repenetrate the stone in each increment a sufficient number of times to develop a significant increase in amperage. *(Minimum increase depends on vibrator used.)*

    f. Stone columns shall be installed so that each completed column is continuous throughout its length.

    g. In the event subsurface obstructions are encountered during construction of a stone column that cannot be penetrated with reasonable effort, construct the stone column following the specified

procedures from the obstruction to the surface. The Engineer may direct the construction of a replacement stone column at another location, which shall be deemed "extra work."

B. The following tolerances shall be observed:

1. Horizontal Control: The center of the completed column shall be within 200 mm (8 in.) of the plan location.

2. Vertical Control: The completed column shall not deviate more than 3 percent of the column diameter from the vertical.

3. Stone Column Diameter: The completed stone column diameter shall not be less than 10 percent below the plan column diameter, unless excessive ground heave occurs due to the presence of unexpected stiff strata of soil. Such heave will waive the column diameter requirements.

   a. If any column falls outside these tolerances, an additional stone column may be required to be installed at the Contractor's expense.

   b. The Engineer may require additional stone columns at the Contractor's expense if the average effective diameter of any group of 50 consecutively installed columns shows that the project post treatment testing requirements have not been met.

4. Finish subgrade to 75-150 mm (3-6 in.) of the plan.

3.2 Method of Measurement and Payment Basis

A. The amount of completed and accepted work, measured as provided above, shall be paid for at the Contract lump sum price for "Stone Columns," which price shall be full compensation for design and for furnishing all materials, and for all labor, equipment, tools, and incidentals necessary to complete the work, including the construction of a working platform.

B. *(A unit price per meter method of measurement may be used.)*

5.4 FIELD INSPECTION AND IMPROVEMENT VERIFICATION[1]

Verification and detailed field inspection of stone 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 stone columns. Furthermore, good communication should be maintained at all times between the inspection personnel, Contractor, Project Engineer and Designer.

a. Stone Column Inspection[27]

This section considers the construction of wet vibro-replacement stone columns.

Important Construction Aspects

1. Inspection records should be carefully analyzed for differences in construction times from one column to the next to both penetrate the ground and construct the stone column. Any significant differences may indicate (1) a change in construction technique, (2) a change in soil properties, or (3) a collapse of the hole. If differences are found, it is important to determine immediately the probable cause.

2. During construction, the probe should be left in the hole at all times and large quantities of water used to help ensure (1) stability of the hole and (2) a clean stone column due to the removal of fines and organics. An average of approximately 11-15 m^3/hr (14-20 yd^3/hr) of water should be used during construction; more water is required during jetting of the hole, with the quantity of water being decreased as the column surface rises.

3. The construction of a strong base at the bottom of the stone column is important to ensure proper performance. Therefore, additional penetrations of the probe are desirable, together with extra care in construction during compaction of the first several increments of stone column backfill. When stone is first placed down the hole, some of it will probably penetrate into the soft clay surrounding the hole near the surface. Therefore, the diameter of the column at the base will not be as large as calculations indicate.

4. The occurrence of unexpected organic and peat layers should be brought to the immediate attention of the Project Engineer and the Designer. The presence of organic and peat layers has been known to cause problems in the performance and construction of stone columns.

As a rule of thumb, the thickness of a peat layer should be no greater than the diameter of the column.

5. If organic soils, such as peat are encountered, effort should be made to flush this material out of the hole; extra flushings are necessary to assure proper removal of the peat. These extra flushings will enlarge the diameter of the hole at this depth and increase the stone take in this area. The stone column should be built as rapidly as possible in peat, silts, and sensitive soils.

6. If localized areas of very soft soils are encountered, it may be desirable to use a coarser backfill gradation.

7. Stone may "hang up" in the hole before it reaches the bottom. To prevent this, and to clean out any soil that may have collapsed into the hole, the probe should be lifted and dropped (stroked) 2-3 m (6-10 ft.) several times after the stone has been added. (Note: if the hole collapses while the probe has been lifted, the probe will not return to the original penetration depth.)

8. Power consumption as defined by ammeter readings is a useful field control that can be continuously monitored. When the specified ammeter reading is reached, this indicates good contact between the probe and the stone. Reaching the specified ammeter reading alone, however, is not a complete guarantee that construction is satisfactory with a high density achieved, and does not replace overall inspection.

9. Achieving the required amp draw on the motor during construction in granular soils is usually no problem; however, obtaining the required ampere draw in soft clays may be difficult. The crane operator can build up misleadingly large amp readings by dropping excessive quantities of stone into the hole and then quickly dropping the probe. Such a practice should not be permitted.

10. In general, larger powered vibrator motors require more current both in the unloaded (free-standing) position or loaded position as they construct a stone column. It is recommended that as a minimum, the free-standing amp reading plus at least 40 additional amps be developed during construction of the column.

11. The inspector should also observe the amount of repenetration after the stone has been introduced. The first repenetration should extend through the newly placed stone, with less penetration occurring on successive repenetrations. Some engineers feel that good repenetration is even more important than the ammeter reading.

b. Inspection Guidelines for Stone Column Construction

The following checklist serves as a general guide for inspection personnel to systematically monitor stone column construction. Items that refer only to the wet method of construction are indicated as such.

I. Stone Column Installation Equipment

The following items are to be checked or noted.

1. Type of stone column equipment as specified in the contract

2. Vibrator characteristics

   a. Diameter of vibrator barrel (mm)
   b. Diameter of vibrator, including stabilizing fins (mm)
   c. Length of vibrator and follower tubes (m)
   d. Power (kW)
   e. Amplitude of free vibration (mm)
   f. Frequency of vibration (rpm)
   g. Eccentric moment
   h. Jets
      (i) Number and location of jets
      (ii) Inside diameter of jets
      (iii) Direction of jet flow

3. Water Supply to Vibrator (for wet feed only)

   a. Pump type and capacity
   b. Supply line type and inside diameter
   c. General condition of water supply line (condition of hoses, leaks, constriction, etc.)
   d. Quantity of water used per hour
   e. Operating pressure

II. Crushed Stone

Following items are to be periodically checked as provided for in the specifications or as considered necessary:

1. Contamination of the stone as it comes from the supplier, including weak aggregate, sand, organics, or other deleterious materials.

2. Gradation of the stone and other applicable requirements, as set forth in the specifications.

3. General contamination of the stone due to the method of stock piling and moving it on site.

III. Working Platform

The following items are to be periodically checked, as provided for in the specifications or as considered necessary in the field:

1. Working platform thickness

2. Gradation of working platform

3. Construction of the platform should be conducted so as to cause a minimum amount of disturbance to the underlying soils. For example, the working platform should be constructed by pushing the sand/aggregate out onto the soft soil from the completed platform using light equipment.

4. If a geosynthetic is required below the working platform, it should meet the project specifications.

IV. Calibration for Quantity of Stone

To permit estimating the in-situ diameter of the stone column after construction, the following data is required:

1. Determine the maximum and minimum density of the backfill, following ASTM Method C29, before stone column construction begins.

2. Determine the volume of the bucket to be used to place the aggregate in the jetted stone column hole.

V. Stone Column Installation:

The following items should be checked or recorded during the installation of each stone column:

1. Record the stone column number and the date and time installation begins.

2. Record the time required to form the hole.

3. Record the stone column length and tip elevation.

4. Observe after jetting that the hole is properly flushed out before the stone is placed. The hole is flushed out by raising and dropping the vibrator at least 3 m as provided in the specifications.

5. Observe that the vibrator is left in the hole during placement.

6. Observe during stone placement that a good upward flow (11-15 m^3/hr (14-20 yd^3/hr) average) of water is maintained at all times to avoid possible collapse of the hole. The upward flow is provided by keeping the jets running on the sides of the vibrator (wet method only).

7. Observe that after the stone is dumped down the hole that the vibrator is raised and lowered a short distance (2-3 m (6-10 ft.)) several times to ensure the stone reaches the bottom and does not arch across the hole; the vibrator should not be completely removed from the hole during stone column installation**.**

8. Estimate that the lift thickness conforms with specifications (normally 600-1000 mm (24-36 in.)).

9. Observe that the vibrator passes through the recently placed lift of stone during the first penetration; additional re-penetrations should have smaller penetration depths into the lift.

10. The specified reading on the ammeter should be developed during the construction of each lift. A continuous record of the ammeter reading may be made by the Contractor. This record should be periodically checked to be sure the equipment operator is satisfying the ammeter requirement based on the specifications.

11. Record the total number of buckets of stone required to construct each stone column. Also, keeping a record of the number of buckets placed in selected lengths of column (and, hence, the quantity of stone used per unit length) permits estimating the approximate diameter of the stone column as a function of depth. Determining the variation of stone column diameter with depth is desirable for obtaining an indication of possible problem strata and the physical mechanics of the construction process. Therefore, for most jobs, the detailed records necessary to define the variation of diameter with depth should be kept during installation of at least the first few stone columns and for selected columns thereafter. If problems are anticipated during installation of subsequent columns, detailed records of stone consumption should be kept for each stone column.

12. Record the total time required to construct each stone column.

13. Carefully observe each stone column after construction and measure the diameter. (Note: Because of low overburden pressure and erosion, the diameter at the surface is generally larger than the average diameter.)

14. Note any unusual phenomenon during or after construction: for example, the subsidence of a stone column, excessive times required to form the hole or construct the stone column, or the presence of underground obstructions. The occurrence of any of these problems or other unusual events should be immediately called to the attention of the Project Engineer.

15. Note the technique, equipment, and adequacy of the method used to penetrate any obstructions. Also note the time to penetrate obstructions.

16. Call the presence of natural gas or unusual odors to the immediate attention of the Project Engineer and the Contractor.

17. Record general comments concerning the adequacy of the overall construction process. Any continuing problems should be brought to the attention of the Project Engineer and the Designer.

VI. Environmental Considerations

Periodically inspect the site to ensure the plans and specifications are being met with regard to all environmental requirements and restrictions, including any siltation ponds, straw or geotextile silt barriers, and general disposal of the effluent from the construction project. Immediately inform the Project Engineer of any problems with meeting environmental site requirements.

VII. General Records

The Inspector and/or Contractor should maintain the following records as the project progresses:

1. A table summarizing the project status, including stone column number, date of construction, stone column length, average diameter, diameter at the surface, total quantity of stone used, total construction time, time to jet the hole, and time to place and densify stone column.

2. A plan of the stone columns, showing as a minimum, the location and number of each stone column, date completed, total quantity of stone used, and total construction time. Each completed stone column should be clearly identified on the drawing.

3. Maintain a record on a weekly basis indicating the general adequacy of the environmental controls and construction progress of the project. Also, periodically take photographs for a permanent record of the site showing the condition of the site with respect to environmental considerations, equipment, and any special features.

Improvement verification of stone columns can be accomplished by visual observation, instrumentation monitoring, and field testing. Observation of ground heave surrounding stone column construction is an indication of soil disturbance. Upward percolation of water to the top of the stone column while adjacent columns are being constructed is an indication of clean construction and high permeability of the stone column.

In granular soils, standard penetration tests and cone penetration tests between columns can be used to evaluate the strength gain in the reinforced soil. Surface loading tests simulating full scale footings can be used to evaluate the column's bearing capacity and settlement behavior. Direct shear tests on completed columns can be used to determine the as-built angle of internal friction of the stone column.

Geotechnical instrumentation is often used to monitor the performance of stone columns during and after construction. Among the instruments commonly used are inclinometers, settlement indicators, and piezometers to evaluate the horizontal and vertical movement of the reinforced soil and its rate of consolidation. Earth pressure cells can also be placed on top of stone columns and at existing ground surfaces between columns to measure the stress ratio in the stone column-reinforced soil.

Vibro-Concrete Columns

The vibro-concrete column (VCC) system can be fully instrumented with an in-cab display to monitor the construction sequence documenting concrete pumping pressure and vibrator power consumption, all related to time and depth. A printout provides a record of the construction of each vibro-concrete column (Figure 23). Testing of the VCC system can also be performed if desired. Load testing of columns can be performed to verify predicted settlements under design loads.

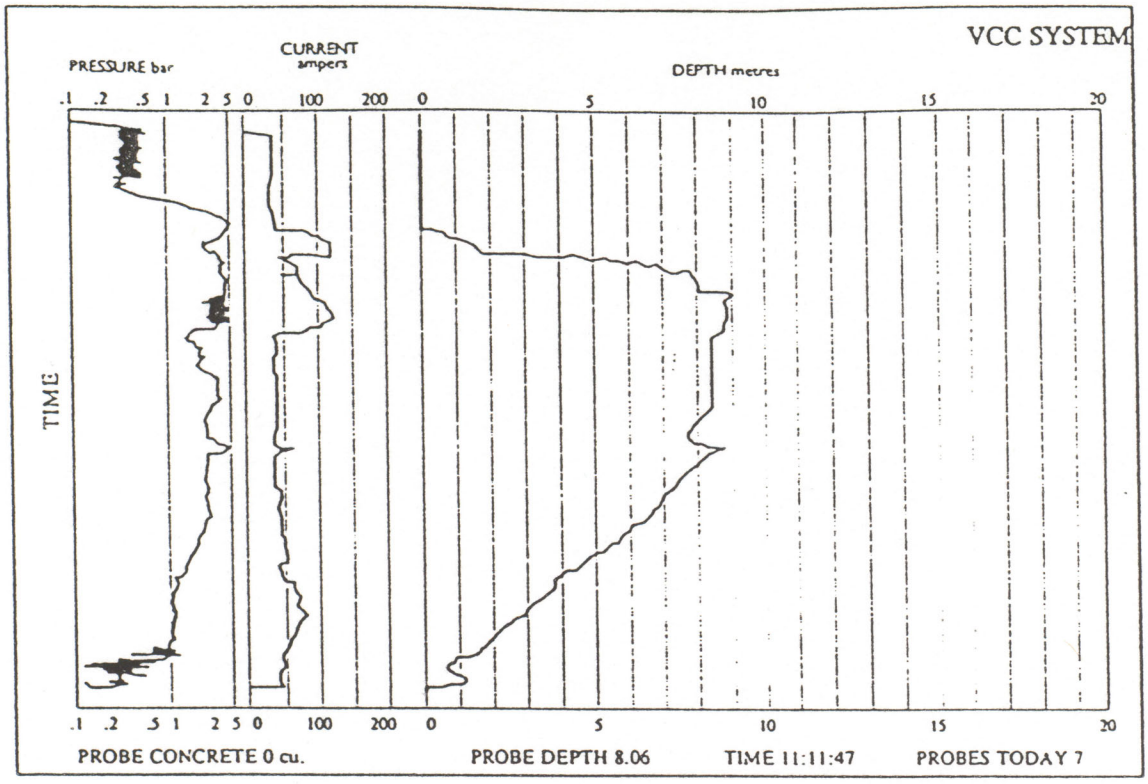


Figure 23. Typical Printout Providing Record of Vibro-Concrete Column Construction.

Geopier Rammed Aggregate Piers

Many of the inspection items for stone columns in connection with excavation, stone quality and quantity, load testing environmental conditions and general records are applicable to rammed aggregate pier construction. A short checklist of quality control items normally performed by the licensed installer and observed by the Engineer are as follows:

- coordinating layout, elevation, and grading
- observing soils encountered during drilling
- measuring drill depths and top elevations of Geopier elements
- controlling moisture content of aggregate within acceptable limits
- controlling and recording type and number of lifts of aggregate
- performing qualitative tests on production pier elements, as appropriate
- implementing corrective measures, when necessary, with approval of the rammed aggregate pier foundation Designer
- reporting construction activities to the Engineer

Unique to rammed aggregate pier field testing is a Bottom Stabilization Test (BSTA), which is a method of verifying that the pier being installed has achieved a general stabilization prior to the completion of the installation. It is also a method to determine that production rammed aggregate piers are comparable in quality to load test piers. This test may be performed on top of the bottom bulb, or, after one or several lifts have been constructed, on top of the bottom bulb. When the compacted aggregate and matrix soil becomes stiff enough to resist downward movement of the tamper, BSTA has been achieved. A pattern of successful BSTA observations is sufficient to reduce BSTA verification to spot checks. The specific procedure for verifying BSTA is as follows:

a. Apply tamper energy to the bottom lift of aggregate and compact aggregate for the same duration and number of passes as in the load test.

b. Turn off the energy source, place a reference bar over the cavity for the pier and mark the tamper shaft at the reference bar.

c. Restart the energy for 15 seconds.

d. Stop the energy and mark the tamper shaft again at the reference bar.

e. The downward movement of the tamper shaft is the distance between the two marks. If displacement is less than 1.5 times the value observed during construction of the modulus load test pier(s), BSTA has been achieved.

f. If BSTA has not been achieved, continue normal construction and repeat steps a-e on successive aggregate lifts.

g. If BSTA is not achieved in the lower 2/3 of the rammed aggregate pier, stop construction, verify the aggregate compaction as described below, redrill the pier cavity, and rebuild the pier, unless directed otherwise by the rammed aggregate pier Designer.

5.5 TESTING

The testing of soils reinforced by stone columns should address the different response of the ground when testing granular 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. Table 2 presents guidance on the usefulness of certain commonly performed test methods.[2]

Short duration tests on metal plates of 600-mm (2 ft.) diameter (small plates in Table 2) are the most common form of testing stone columns in 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 stone column behavior, particularly when residual porewater pressures are present in the ground.

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 stone 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 stone 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 stone column.

Table 2. Suitability for testing stone columns.

Test	Granular	Cohesive	Comments
Dynamic Cone	**	*	Too insensitive to reveal clay lenses. Can locate dense layers and buried features.
Mechanical Cone	***	*	Rarely used.
Electric Cone	****	**	Particle size important. Can be affected by lateral earth pressures generated by treatment. Best test for seismic liquefaction evaluation.
Boreholes + SPT	***	**	Efficiency of test important. Recovers samples.
Dilatometer	***	*	Rarely used.
Pressuremeter	***	*	Rarely used.
Small Plate Load Test	*	*	Does not adequately confine stone column. Affected by pore water pressures.
Large Plate Load Test	**	**	Better confining action.
Zone Loading	****	****	Best test for realistic comparison with foundations.
Full-Scale	*****	*****	Rare

** least suitable ***** most suitable*

CHAPTER 6
COST DATA

This chapter presents guidelines for preparing budget estimates in order to determine the economic feasibility of stone columns. There are many factors affecting the price of stone 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 stone columns be contacted to verify both the budget cost calculations and the technical feasibility of stone column installation.

Typical stone column highway projects have been reviewed to determine the cost range of projects in North America where stone columns have been utilized. The cost data was developed from a list of 125 projects that have been successfully completed by experienced contractors between 1984 and 1994.

Budget estimates can be prepared based on the information developed and provided in this chapter.

Using the information in the preceding chapters, a determination can be readily made as to the depth of the stone 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 stone columns. It is recommended that for stone 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. Basic Budget Estimate for Stone Column Installation

A budget estimate would typically include the following elements:

Mobilization/Demobilization

Mobilization/demobilization costs will depend on the number of rigs required to complete the work on schedule, the type of crane needed to support the vibrators, and the distance required for the equipment to be transported. As a minimum, the mobilization/demobilization cost will be $15,000 (1998) per rig.

Cost of Stone Column Installation

The basic cost of stone column installation is calculated by

1) Determine the required number of stone columns and dividing the square meter spacing for each stone column into the total treatment area

2) Multiplying the number of stone columns by the depth of treatment

The material cost of the stone backfill is a major component of the project and can account for over 40 percent of the estimated cost of stone column installation. The cost of stone backfill will vary from site to site. Where suitable backfill material is not readily available, the increased transportation costs may also affect the stone column installation pricing.

The minimum cost for vibro-replacement stone column installation, *based on readily available suitable backfill material*, is $45 per linear meter ($14/lft.), with a dry vibro displacement stone column starting at $60 per meter ($18/lft.).

Additional Costs

To this cost per linear meter of stone column installation, the cost of mobilization/demobilization, site specific load tests (if required), inspection, verification testing, etc., are all added.

b. Sample Budget

A budget example for a highway embankment foundation support for a highway follows:

Subsurface Conditions and Project Area

Assume layers of loose sands and soft silts to a depth of approximately 10 meters.

Assume an overall treatment length of 300 meters, with a bottom width of the proposed embankment of 50 meters.

Consider that the influence of the embankment fill would be 10 meters outside the embankment's dimensions, based on the 45-degree rule of thumb. The actual treated area would be 320 meters by 70 meters or 22,400 square meters.

Stone Column Design and Quantities

Assume that design calculations indicate that the desired results can be obtained by placing the stone columns on a 2.5-meter square-grid pattern.

Each stone column would, therefore, account for 6.25 square meters of treated area, equating to the installation of 3,584 stone columns.

Installation to a depth of 10 meters would equate to 35,840 linear meters of stone column.

Cost of Stone Column Installation

Assuming that environmental constraints would require the dry bottom feed vibro displacement technique at an estimated cost of $60 per linear meter, the installation cost per stone column would be $600 and the total cost of the installation would be $2,150,400. To this figure would be added mobilization/demobilization, testing, inspection, etc.

c. Vibro-Concrete Columns

Budget costs for vibro-concrete columns are similarly calculated. However, the cost of pumped concrete raises the minimum cost of the column to $75 per meter.

d. Geopier Rammed Aggregate Piers

At present, rammed aggregate piers are more typically used for commercial structure foundation support. Limited information indicates that they are typically bid at $250 to $750 per pier.

Based on the low end lump sum cost per pier (including gravel) and a typical 760-mm (30 in.) diameter pier at a maximum depth of 3.5 m (11.5 ft.), a minimum linear meter cost of $75 is indicated. On a volume in-place basis, a cost of $150 per m^3 can be computed. Mobilization costs vary from $5,000 to $15,000, and modulus tests generally run around $5,000/test.

CHAPTER 7
CASE HISTORIES

Representative case histories of transportation-related construction projects are presented to illustrate the application of stone column technology.

7.1 SLOPE STABILITY IMPROVEMENT

NYDOT Route 22, Wadhams, NY[8]

Along a 67-m (220 ft.) length of New York Route 22, slow, continuous movements over a 10-year period had created a constant and expensive maintenance problem. The failure extended from the centerline of the roadway into a swampy area 40 m (130 ft.) down slope. Of particular importance in addressing slope stability improvement alternatives was that the site lay within the Adirondack State Park and within 15 m (50 ft.) of a registered wetland. Selection of the method of improvement was, therefore, governed by environmental considerations.

Borings conducted at the failure location revealed a 3-meter-thick layer of silty clay overlying a 3- to 6-m (10-20 ft.) layer of over-consolidated, soft, silty clay. Beneath this clay layer was a silty gravel in which an artesian head was encountered. Geotechnical laboratory tests showed that the liquidity index and activity of the clay were 1.0 and 0.5, respectively.

Three potential solutions were analyzed: stabilizing berm, shear key, and stone columns. Berm treatment would require additional right-of-way in the wetland area, and the shear key alternative would require a wide and deep supported excavation. Both options would be prohibitively expensive. Stone column installation by the dry, bottom-feed method was selected as being technically feasible, environmentally acceptable, and economically advantageous. The stone columns would be installed through the soft clays into the gravel layer to intercept the slip plane near the gravel/clay interface at a depth of 4.8 m (16 ft.).

A minimum 0.3-m-thick (1 ft.) drainage blanket was placed over the work area prior to stone column construction. Concern that column installation would alter the in-situ stresses of the soil and increase pore water pressure required careful evaluation of installation sequencing. As a result, columns were installed from the bottom of the slope up. The arching effect of the stone column/soil interaction would allow much of the downward driving force to be carried by the columns, preventing additional soil from being displaced downhill. Excess

pore water pressure would dissipate through the vertical drainage path provided by the columns.

Figure 24 illustrates the construction on the slope.

Following a five-column test section to establish acceptance criteria, production columns were installed to a maximum depth of 10.7 m (35 ft.) through the clay layer and into the silty gravel. During initial production, piezometer and inclinometer readings verified that pore pressure build-up was local and rapidly dissipated and that slope movement was minor. This allowed the installation sequence to be reappraised. Subsequent columns were installed in a cost effective manner that still met acceptance criteria.

Prior to production, slope movement was measured at approximately 0.08 mm (0.003 in.) per day. Over the course of the stone column installation, total additional movement was 3.3 mm (0.13 in.). As of May 1995, some eight years after project completion, New York D.O.T. reported little to no movement recorded.

Figure 24. Construction of Stone Columns, Rt. 22, NY.

7.2 LIQUEFACTION RETROFIT[28]

7th Street Terminal, Port of Oakland, CA

The Seventh Street Marine Terminal at the Port of Oakland, California, is situated on the San Francisco Bay near the eastern end of the Oakland-San Francisco Bay Bridge and is constructed on land reclaimed from the Bay in the 1960s. This area contains several major faults, including the San Andreas, Hayward, San Gregorio, Calaveras, and Healdsburg-Rodgers Creek. During the historical period (160 years), these faults have produced 10 moderate-to-large earthquakes affecting the Bay Area.

On October 17th, 1989, the Loma Prieta earthquake emanated from a portion of the San Andreas fault in the southern Santa Cruz Mountains, approximately 80 km (50 miles) south of the Terminal site. Ground shaking recorded at the Port's Berth 25 Wharf reached a horizontal peak ground acceleration of 0.29 g. Following the earthquake, effects of strong ground shaking were evident at the Terminal in the form of liquefaction sand boils, lateral spreading fissures, and differential settlement. Significant damage was sustained by the Terminal wharf structure, due primarily to liquefaction-related lateral spreading and settlement of hydraulically-placed sand fill materials and native silty sands beneath and behind the perimeter rockfill dike. The perimeter dike moved bayward approximately 0.3 m (1 ft.), and many of the 405-mm (16-in.) square prestressed concrete footings bearing on fill settled 225-350 mm (9-14 in.). The terminal was closed for approximately six months and operated with limited crane capability for 22 months after the earthquake while seismic repair work proceeded.

Based on the observed dike performance, ground-shaking from future earthquakes was expected to affect the stability of the perimeter dike primarily by seismic inertia forces within the dike and by shaking-induced strength reductions in the hydraulic sand fills and upper native sands. To mitigate the liquefaction hazard and minimize the risk of future lateral spreading or dike failures, alternatives were examined for stabilizing the perimeter dike by strengthening a zone of soil within the dike fill and upper native sand deposits below the dike. Stabilization alternatives examined included grouting, stone columns, and compaction piles. Dike performance evaluations indicated that a zone of stabilization approximately 12 m (40 ft.) wide and extending in depth to at least the bottom of the upper native sands would be required to achieve the Port's performance criterion that dike movements be limited to a few centimeters during future earthquakes. Because of these considerations and cost, the installation of stone columns using vibro-replacement was recommended to achieve the desired characteristics. By increasing the density of the loose-to-medium-dense sand materials, replacing some of those sands with compacted stone columns, and providing a

drainage path to dissipate excess pore water pressures generated during earthquakes, the stabilizing effects could be achieved. The specifications for the stone columns included diameters of 0.9 m (3 ft.) spaced 2.4 m (8 ft.) apart. The ground improvement scheme developed for the wharf is shown in Figure 25 and a project photo in Figure 26.

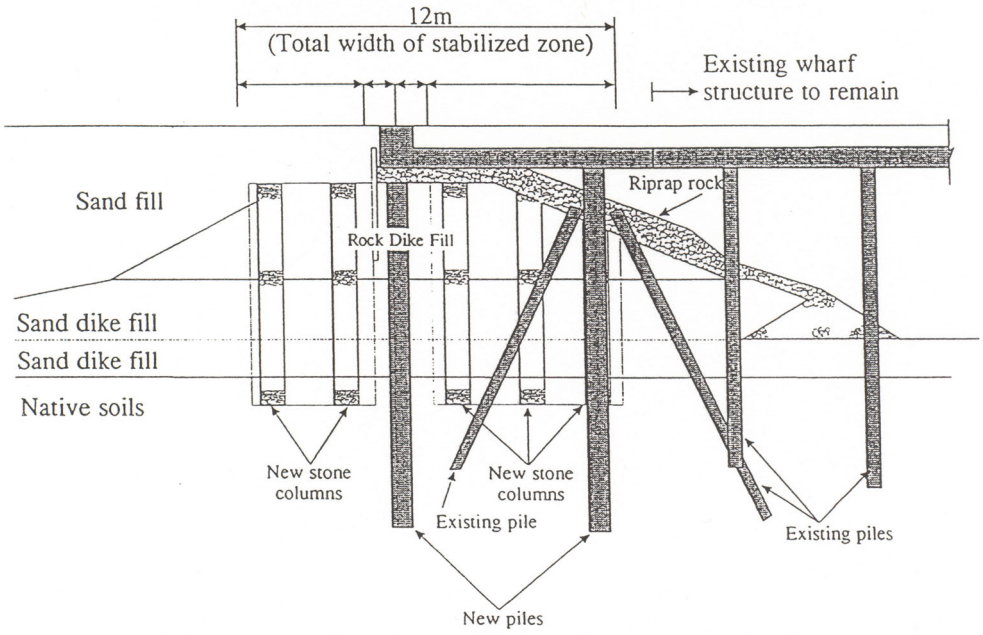


Figure 25. Stone Column and New Piling for Port of Oakland Wharf Repair.

Figure 26. Construction of Stone Columns, Port of Oakland.

The stone column process effectively densified subsurface materials along the entire reach of the wharf. Extensive post-construction Standard Penetration Testing verified that the vibro treatment improved the penetration resistance of the loose sand, as measured by blow count values (N), by as much as 400 percent, as illustrated in Figure 27 where the open circles represent the initial conditions.

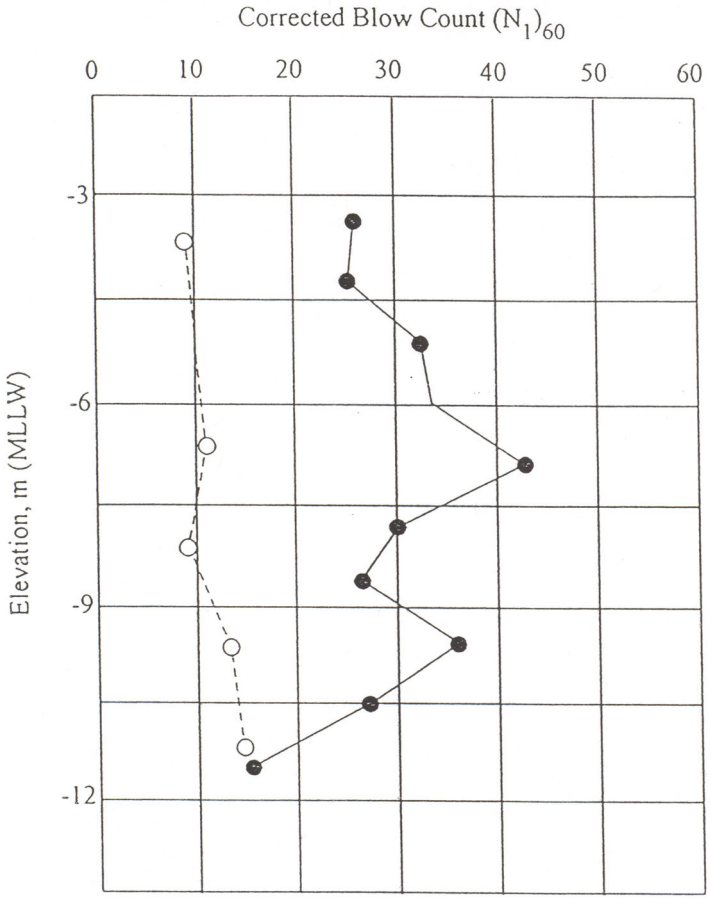


Figure 27. Standard Penetration Test resistance Values for Port of Oakland Wharf Repair.

7.3 OVERALL STABILITY

SR 500-Andresen Road, Vancouver, WA

The Andresen Road grade separation structure for SR 500 in Vancouver, WA, is within a seismically active area that has a design acceleration coefficient of 0.15 g. Geotechnical considerations for WSDOT engineers included settlement of the bridge structure and approach embankment fills, as well as soil liquefaction.

Subsurface investigations by WSDOT geotechnical engineers revealed variable, loose-to-medium-dense sands to depths of 6 m (20 ft.). Underlying the loose, liquefiable sands and extending to depths of 24 m (80 ft.) was medium-dense-to-dense sand containing greater amounts of silt and clayey silt. Groundwater was located at depths of 1.5-2.4 m (5-8 ft.).

The bridge, a 57.3-m-long (190 ft.) by 28.3-m-wide (93 ft.) single-span, post-tensioned, concrete box girder, is supported by 8.2-m-tall (27 ft.) abutments at each end. Tapered retaining walls project at 45 degrees from each abutment end to support new embankments. The four wing walls are approximately 30.5 m (100 ft.) long, range in height from 0.6 m (2 ft.) near the toe of the embankments to 8.2 m (27 ft.).

With settlement and liquefaction governing the foundation design, two options were considered as support for the abutments and retaining walls: standard cast-in-place displacement piles, and shallow spread footings.

Since ground modification was required for both options, WSDOT elected to use shallow footings supported on stone columns, as they offered significant cost savings. The stone columns were selected to mitigate potential liquefaction and to reduce post-construction settlement of the abutments to less than 12.7 mm (0.50 in.). With the stone column support, shallow spread footings were designed with 216 kPa (4500 psf) bearing pressure.

The vibro treatment was provided at each bridge end, as shown in Figure 28, effectively treating an area measuring 38.4 m by 42.7 m (126 ft. by 140 ft.) per bridge end. This was large enough to provide improvement for the bridge abutments and all four of the approach retaining walls. A maximum treatment depth of 6.1 m (20 ft.) was used in order to penetrate a 1.5-m-thick (5 ft.) working platform and extend to the bottom of the 4.6-m-thick (15 ft.) liquefiable layer. The dry, bottom feed method of vibro-displacement was utilized with a 123 kW "S" type depth vibrator.

The specifications called for densification of loose, sandy soils to a minimum Standard Penetration Test "N" resistance of 27 blows per 300 mm (12 in.) or an average per test hole of 98 blows per meter (30 per ft.). A plate load test was allowed as an alternative test procedure to verify settlement reduction (Figure 29).

Pre-treatment testing results indicated ranges of SPT "N" values from 6 to 95 blows per meter, with an average "N" of 33 blows per meter. Post-treatment testing in the middle of the treatment pattern ranged from 46 to 253 blows per meter, with an average of 154 blows per meter. The minimum "N" value of 46 blows per meter was found acceptable, as it was

taken in sandy silts that were less susceptible to liquefaction. On average, the vibro-replacement stone columns achieved a densification improvement factor of 470 percent as measured by the SPT.

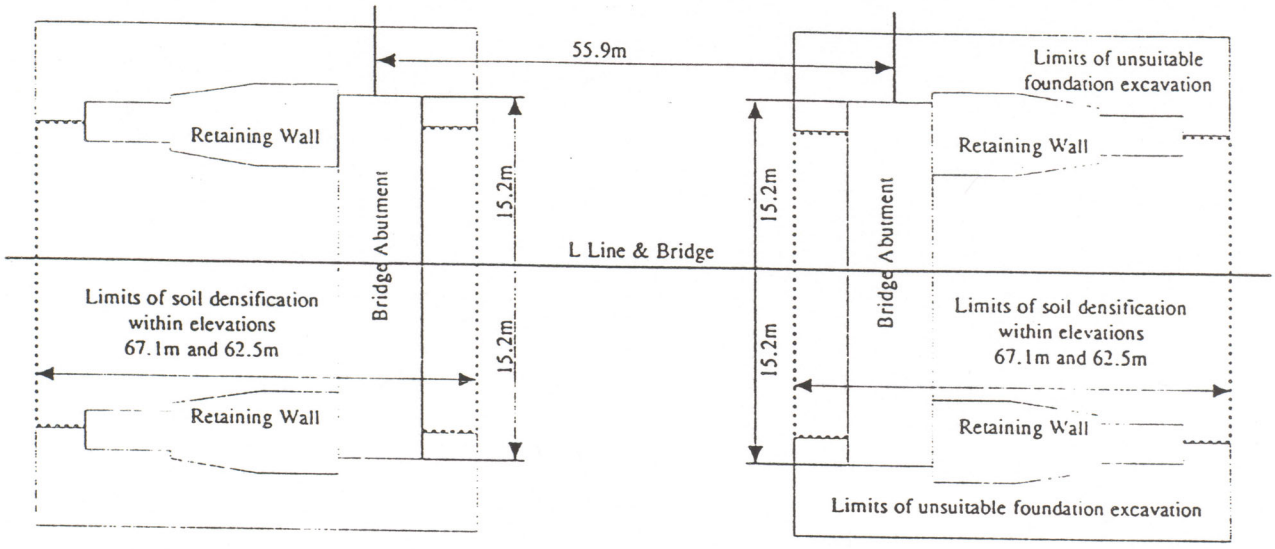


Figure 28. Vibro-Replacement Provided at Each Bridge End for Andresen Road.

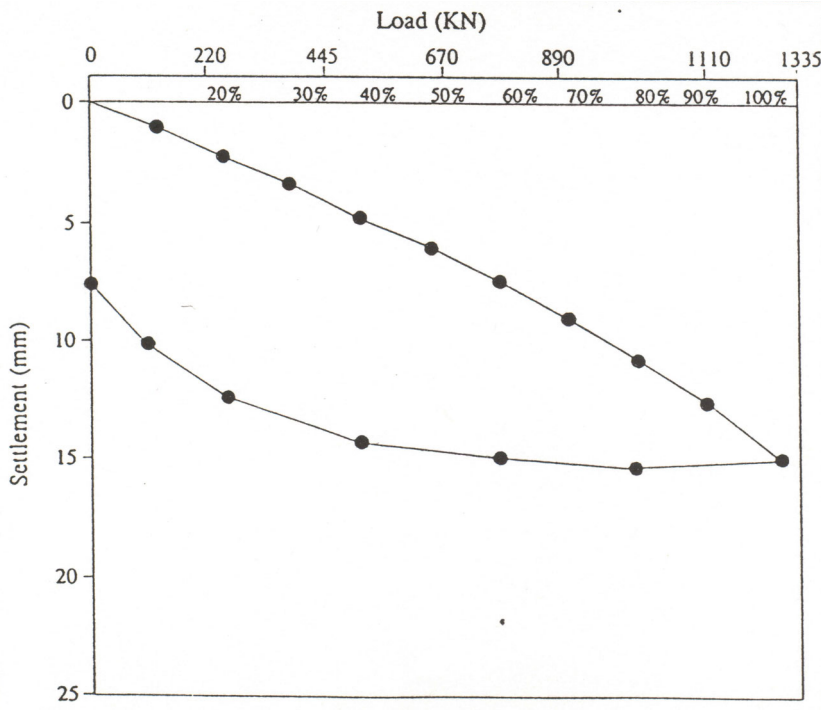


Figure 29. Plate Load Test to Verify Settlement Reduction of Andresen Road.

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Technical Summary # 6

DEEP SOIL MIXING

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CHAPTER 1
DESCRIPTION AND HISTORY

Soil mixing for deep soil stabilization comprises ground improvement techniques that mix reagents with soils at depth to improve soil properties in-situ, without excavation or removal. In the United States, these methods are generically referred to as Deep Mixing Methods (DMM). The mechanical mixing aspects of in-situ soils and reagents distinguish this ground improvement method from others. DMM can be utilized for groundwater cut-off, excavation support, soil stabilization, settlement reduction, foundation support, liquefaction risk mitigation, and the fixation of contaminated soils. Surface mixing with stabilizers for pavement subgrade improvement is considered outside the scope of this technical summary.

DMM is performed under a number of different names or acronyms worldwide, many of them proprietary. Although the basic concept and procedures are similar for all techniques, the mixed soil product and the objectives of the soil mixing program can be divided generically by the use of standard acronyms, as recently suggested by Bruce, on the following basis:[1]

- Method of additive or binder injection. Use (W) for wet, or (D) if dry injected.
- Method by which additive is mixed. Use (R) if by rotary energy, or (J) if the mixing is enhanced/facilitated by a high-pressure jet.
- Location of mixing action. Use (E) if near the drilling tool, or (S) if along shaft for a significant distance above via augers and/or paddles.

The classification shown on Figure 1 was therefore proposed, and four categories – WRS, WRE, WJE and DRE – have been identified. No methods have been found in the DRS, DJE, or DJS categories, since dry injection methods only feature end mixing with relatively low pressure binder injection pressures via compressed air, and jetted methods only feature end mixing (hence no WJS).

This technical summary will focus on the two most widely used categories, which fall into two distinct groups as follows:

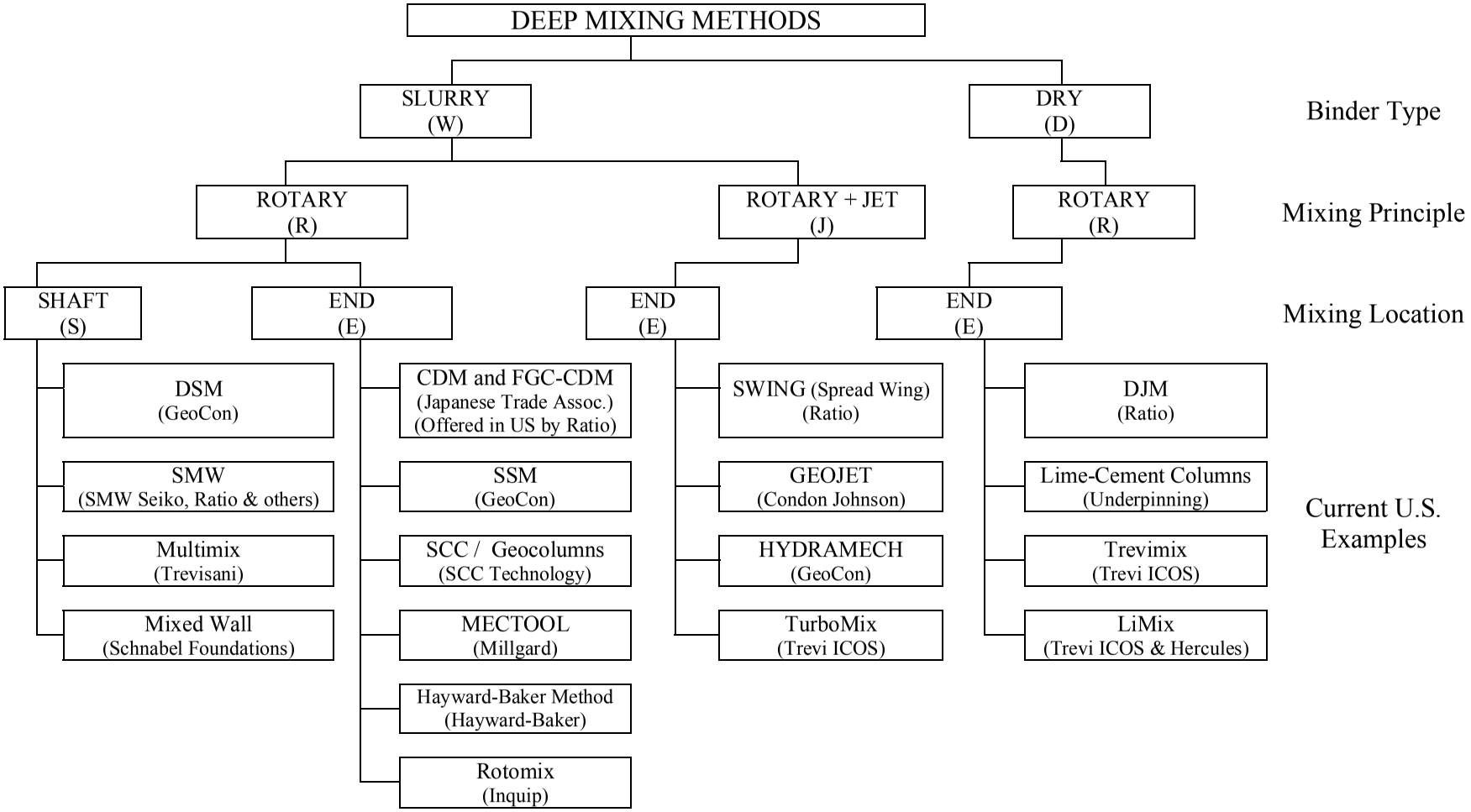

Figure 1. Generic Classification of Soil Mixing Techniques. [19]

Wet Deep Mixing Methods (WRS, WRE, WJE)	***refers to wet***, single or multi-auger, block or wall developed for large-scale foundation improvement and containment in any soil. Primary reagents are cement-based grouts.
Dry Deep Mixing Methods (DRE)	***refers to dry***, single-auger column technique developed for soil stabilization and reinforcement of cohesive soils. Primary additive is granular or powdered lime for lime columns and cement or lime-cement mixtures.

For both of the above categories, other additives may be incorporated, as discussed in the Materials section of this summary.

1.1 HISTORICAL OVERVIEW

The first soil improvement technique utilizing mechanical equipment to mix cementitious materials with in-situ soil was the Mixed-in-Place Piling technique. Intrusion Prepakt in Cleveland, Ohio, developed this technique in the United States in 1956. In this process, a mixing head, supported by a square, hollow shaft, is rotated into the soil, as cement grout is injected through the hollow shaft and out of the injection tips in the mixing head during its downward travel. Rotation continues with grout injected, as the mixing head is withdrawn, thereby assuring complete mixing of soil and grout.

The principal advantages of Mixed-in-Place Piling were two-fold. Since the existing soil was being used, it was a more economical process than other piling techniques, and the mixed-in-place piles could be completely interlocked to form watertight cut-off walls. This system was successfully used for cut-off walls beneath dams, beachfront retaining walls, and retaining walls for excavations. Cost was competitive to sheet piling, and the cut-off of subsurface water was effective. The compressive strength of the soil-cement mixture was dependent on the type of soil encountered. In clean sands and gravels, typical unconfined compressive strengths of 31 MPa (4500 psi) were obtained. In silt or silty sand, strength was less than 7 MPa (1000 psi). The process was normally not used in clayey soil because of the difficulty of obtaining a uniform clay-grout mixture. This patented system was used for over thirty projects throughout the United States, but the equipment was not effective in coarse gravel, or where boulders and other obstructions were encountered. In Japan, however, the mixing technique gained wide acceptance.

In the early 1960s in Japan, the Port and Harbor Institute of the Ministry of Transport undertook research and development of deep soil stabilization methods using granular or powdered lime as stabilizing agents for ground improvement in the construction of port facilities on alluvial clay and for sea reclamation. This concept was put into practice in 1974 using quicklime. In this method, quicklime is compacted into a predrilled hole and expands during the slaking of the lime, which consolidates the soil between columns. As the volume increases, the water content of the soft soil around the holes is significantly reduced. Since this method relies on strengthening of the soil by both compaction and reinforcement, it works best in soft, cohesive soils. The reduction of water content in this type of soil has a significant effect on the shear strength and bearing capacity. This method has also been extensively used in India, China, Singapore, Malaysia, and Taiwan.[2]

In 1975, soil mixing using cement grout was introduced in Japan for large-scale stabilization of marine clays for land reclamation.

At the same time, independent research in Scandinavia produced a lime column technique for the stabilization and reinforcement of very soft, cohesive soils. This technique differs from the Japanese approach to lime columns in that the Scandinavian column is mixed in-situ. Over five million linear meters of lime columns have been installed in Sweden since 1975, and the current production of lime and/or lime-cement columns in Sweden and Finland ranges between 3 – 4 million meters per year.[1]

Since the mid 1990's in Sweden, extensive use has been made of lime-cement columns for deep stabilization of soft soils, mainly for the reduction of settlements and improvement of stability for the construction of new roads and railroads.[4] As with the previously described lime column, the lime-cement column is also constructed using dry reagents.

Although it had become widely accepted overseas, deep soil stabilization did not gain general acceptance in the United States because it was not, at that time, cost effective compared with conventional technology. Since then, significant Japanese advances have been made to the basic system by the mounting of multiple auger shafts to increase productivity and by the development of more powerful motors to overcome obstructions. With the availability of appropriate equipment, soil mixing has become a viable method in the American construction market. In 1987, the first United States use of soil mixing using purpose-built equipment was a liquefaction mitigation project for the Bureau of Reclamation. Hexagonal, soil-cement cells were constructed through the foundation beneath the Jackson Lake Dam in Wyoming to contain the soils in the event of an earthquake. It is believed that the first highway application in the United States was in connection with retaining wall construction at Lake Parkway in Wisconsin. Most recently, this technique has been used extensively on

the Boston Central Artery project for excavation support, toe stabilization required for cut and cover tunneling, cutoff walls, and as a stability buttress; in Minnesota, to limit settlements beneath a three-span bridge and approach ramps walls; and to stabilize embankments on the Woodrow Wilson Bridge project.

Lime column technology has been discussed in United States technical literature since 1977. However, the first lime column system was not installed until 1992 by the Florida Institute of Phosphate Research to determine if this technique could be of use in recovering waste clay ponds. Most recently, use of lime-cement technology has been made in connection with the reconstruction of I-15 in Salt Lake City. The lime-cement columns have been used to enhance embankment stability and decrease settlement.[5]

1.2 FOCUS AND SCOPE

Deep mixing methods are a group of ground improvement methods that use either wet or dry reagents and utilize mechanical equipment to mix cementitious materials with the in-situ soil. For the purposes of this technical summary, deep mixing methods (DMM) refer to both the wet method of construction using primarily cement-based grout reagents, and to lime or lime-cement columns (LC, CC, or LCC), the dry method of construction. Other reagents may be included for site-specific subsurface conditions, and these are addressed as well.

The focus and scope of this technical summary is to identify problems that have been successfully solved by these methods and to synthesize the current state-of-the-practice of soil mixing design and construction. This summary will address design considerations, construction and materials, design concepts, bidding methods, construction specifications, and construction control. 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. Case histories are presented to illustrate DMM projects utilizing wet and dry reagents.

This technical summary is not intended to be a standard design and construction reference on deep soil stabilization. The emerging nature of this technology and limited United States experience to date (2003) result in the current absence of a comprehensive technical reference and cost information analysis.

1.3 RECOMMENDED READING

Since deep soil stabilization is an emerging technology, the availability of comprehensive technical literature in the United States is limited. Literature that is available typically considers some aspect of the state-of-the-practice and is cited frequently in this technical summary. The following sources in particular are referred to extensively:

- Broms, B., "Lime Stabilization." Ground Improvement.[2]

- Taki, O and Yang, D.S., **"**Excavation Support and Groundwater Control Using Soil-Cement Mixing Walls for Subway Projects."[10]

- Yonekura, R., Terashi, M., Shibakazi, M. (Editors) (1996), "Grouting and Deep Mixing," Proceedings of IS-Tokyo 96/The Second International Conference on Ground Improvement Geosystems. Tokyo, Balkema.[6]

- Bruce, D., "An Introduction to the Deep Soil Mixing Method used in Geotechnical Applications, Verification and Properties of Treated Ground," FHWA RD-99-167.[1]

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CHAPTER 2
DESIGN CONSIDERATIONS

As with other emerging technologies, the applications of DMM continue to expand with every completed project. This chapter discusses applications, advantages and disadvantages/limitations, applicable soil types, and the steps required to develop a feasibility evaluation, as well as addressing competing conventional technologies.

2.1 APPLICATIONS

Wet DMM were developed primarily for large-scale structural support and containment, while the dry lime-cement column technique was developed primarily for soil stabilization/reinforcement and settlement reduction. However, as with many ground improvement technologies, the original concepts have been expanded since initial development to encompass a broad range of applications, as subsequently described. Figure 2 gives an overview of typical applications recently identified.[3]

a. Wet Mixing

The current applications are discussed below.

Foundation Improvement

Wet DMM are used extensively in Japan for large-scale land and underwater soil improvement to provide a stable foundation capable of bearing the structural loads and for seismic stabilization of waterfront developments. In the United States, large volumes of foundation soil have been strengthened in conjunction with deep excavations and structural foundations at the Fort Point Channel crossing for the Central Artery tunnel project in Boston; at the Port of Oakland; and for a bridge and abutments in Minnesota. The most significant typical usage is for settlement control and/or shear strength improvement under embankments. Under the latter usage, DMM columns are typically constructed in grid or lattice geometry to provide additional resistance to bending.

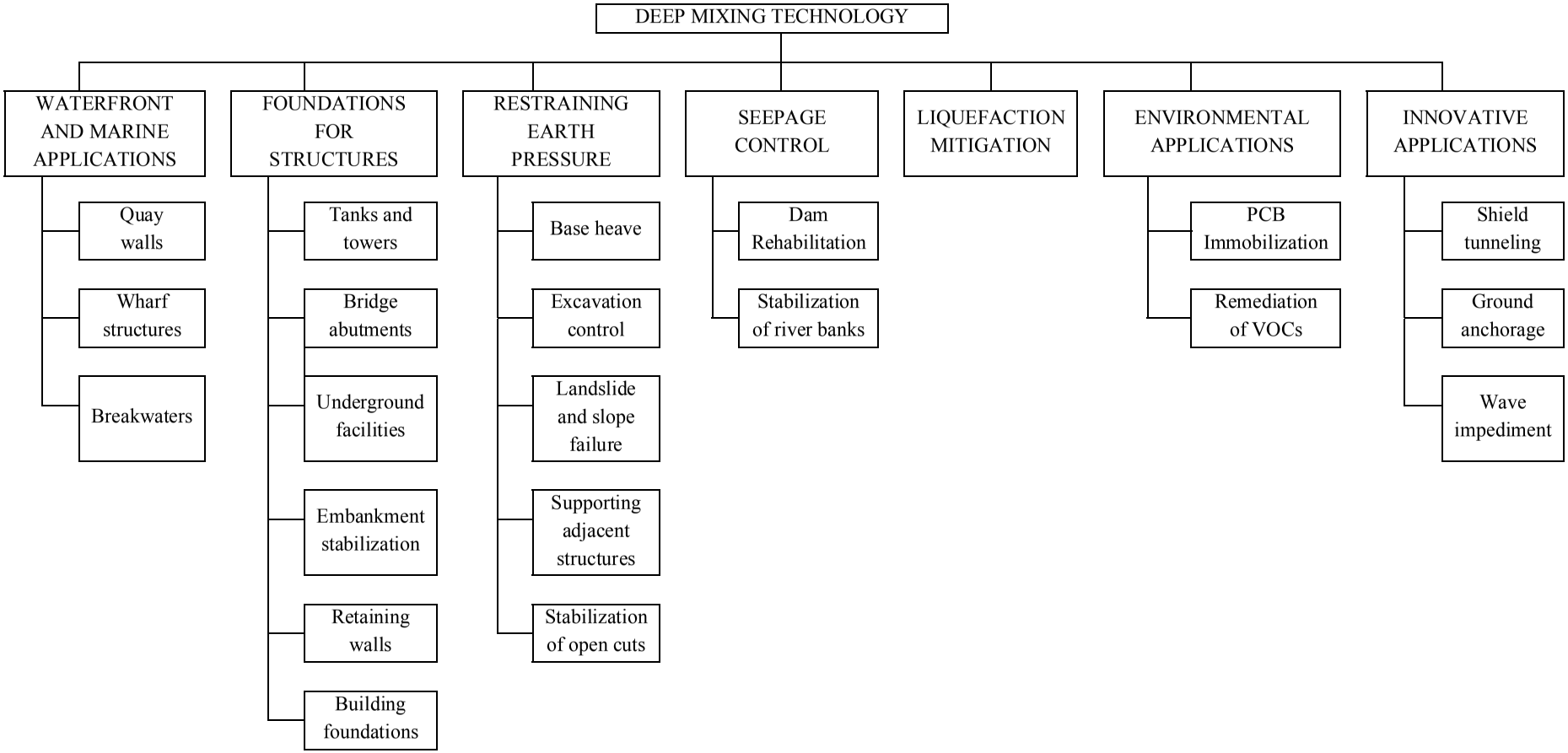

Figure 2. Range of Applications for Deep Soil Mixing.[3]

Liquefaction Risk Mitigation

DMM can be used to improve mass shear strength and contain liquefaction propagation. The first use of DMM for liquefaction mitigation with multi-stem mixing tools in the United States was for the Bureau of Reclamation's Jackson Lake Dam in Wyoming in 1987. The dam was originally built of hydraulic fill on alluvium and outwash in 1917. Seismic analyses performed by the Bureau determined that the dam and its foundation would be susceptible to liquefaction during the upgraded design earthquake.[8] To control liquefaction of the foundation soils, a series of six-sided cells were constructed through the foundation to contain the soil in case of an earthquake, thus preventing a shear failure.

Conventional alternatives to DMM for liquefaction mitigation include vibro-compaction and stone columns.

Excavation Support Walls

DMM walls have the capability to form continuous, low permeability support walls for deep excavations. These walls can accommodate large, steel H-beams placed in every auger hole or in any sequence, as required by wall loadings. The soil-cement in the cylinder acts like lagging between the beams and forms a vertical barrier to groundwater. These walls can be designed as earth retention walls, using internal cross bracing or with soil and/or rock anchors with necessary walers. Conventional alternatives include soldier piles and lagging, sheet piles, and slurry diaphragm walls. Recent use of this application was in connection with excavation support in Wisconsin at Lake Parkway and more recently on a number of projects at the Central Artery in Boston, Massachusetts, in connection with excavation support, cutoff, and slide buttress applications.

Cut-Off Walls

DMM wall configurations can be constructed to depths in excess of 30 m (100 ft.) to serve as groundwater or containment barriers. Laboratory tests of field samples from a soil mixed wall have shown coefficients of permeability ranging from 10^{-4} to 10^{-7} cm/sec. Conventional alternatives include soil-bentonite, cement-bentonite, and jet grouted cut-off walls.

Hazardous Waste Containment

Solidification/containment technologies are designed to modify the treated material by improving the handling and physical characteristics of the waste, decreasing its surface area, limiting the solubility of its hazardous constituents, and/or detoxifying the contained

contaminants. Stabilization/solidification (S/S) technologies, such as soil mixing, are sometimes cost effective methods for the treatment and/or immobilization of hazardous wastes and contaminated soils.

In-situ stabilization techniques have been in use since around 1980. The primary application has been in the stabilization of soft sludge and wastes in old lagoons. The use of auger and caisson drill equipment has been adapted to in-situ stabilization of wastes and contaminated soils having a variety of consistencies. Advantages of using this in-situ stabilization drilling technique include the following:

- Dewatering is not required.
- Fugitive emissions can be controlled using optional dust collector equipment.
- The reagent injection rate can be controlled and blended with the material to be stabilized.

b. Dry Mixing (Lime, Cement, and Lime-Cement Columns)

Lime columns were designed and used extensively in Sweden and Finland mainly to improve soft, cohesive soils. Subsequently, lime-cement mixtures were introduced and have become the standard. Lime-cement columns (LCC) are typically used for the following.

Settlement Control

Both total and differential settlement can be reduced with lime-cement columns, using the same rationale as for stone columns. The lime-cement columns being stiffer and relatively less compressible than the adjoining soil can carry a greater portion of the load, thereby reducing both total and differential settlement. The amount of reduction is a function of the area replacement ratio and the stress concentration factor, which is a function of the relative stiffness of column to untreated soil. With respect to time-rate of settlement, the concept and calculations for lime columns are similar to those for wick drain projects, provided that the permeability of the columns are at least one order of magnitude less than the native soil. Lime columns can meet these criteria, but lime-cement columns typically may not. Some increases in the rate of consolidation are claimed for LCC, but they have not been sufficiently demonstrated to date. Conventional alternatives include wick drains, stone columns, and surcharging.

The use of dry mix techniques using cement only to reduce settlement and increase global foundation strength at bridge approaches has been recently documented for the Fu-Xia Expressway in southeast China, where approximately 18 km of roadways were constructed

over soft marine clays. The alignment contained numerous bridges, underpasses, and overpasses. The project was opened to traffic in 1997.[7]

Improved Stability

LCC can be used to reinforce existing soils by increasing their mass shear strength. Thus, the stability of embankments, slopes, trenches, and deep cuts can be improved. The use of LCC to improve foundation stability and reduce settlement under roadways has been extensively used in Scandinavia. Typically, they are placed in a grid pattern under the embankment and in interconnected rows under the slope to provide sufficient resistance to bending.

Alternative methods could include wet mix methods, stone columns, surcharge with stage loading, and deep foundations.

Increasing Sheet Pile Wall Stability

Lime or cement columns can be used to increase the stability of an anchored sheet pile wall. The columns will increase the passive earth pressure at the toe of the wall during construction and they will also reduce the settlements of footings constructed on the stabilized soil inside the excavation. LC or LCC can also be placed behind the wall in order to reduce the lateral earth pressure acting on the sheet piles, since the columns increase the average shear strength of the clay behind the wall.

2.2 ADVANTAGES AND DISADVANTAGES/LIMITATIONS

a. Wet Deep Mixing Methods (DMM)

Advantages

- DMM can be accomplished to depths up to 70 m (230 ft.) for marine work and up to 30 m (100 ft.) for land operations.

- It is conceptually applicable for most subsurface conditions, from soft ground to denser soils, with materials varying from plastic clay to sand and gravel with cobbles. However, its chief application is the improvement of soft cohesive soils and loose to medium-dense non-cohesive soils.

- Since the DMM technique utilizes the existing soil, it can be more economical than removal and replacement. The problems associated with disposal of the waste material are considerably reduced in an amount proportional to the percentage of additives used and the moisture content of the in-situ soils.

- Costly dewatering is not required with DMM.

- The construction is a drilling process which is ideal in noise- and vibration-sensitive areas.

- Due to the use of multi-axis augers and the use of in-situ soil, the production rate for wall construction is higher than that of concrete slurry walls or other underground walls.

Disadvantages/Limitations

- The relatively high cost of mobilization/demobilization of the rigs to support the multi-auger or the large-diameter auger systems, plus the cost of the accompanying auxiliary batch plant makes these systems uneconomical for small construction projects.

- As the existing soil is to be utilized in-situ, a thorough geotechnical testing program will be required. Special requirements will include some continuous sampling, and laboratory testing of the soil-reagent mixture to determine preliminary strength, modulus, permeability, or other design parameters, such as freeze-thaw durability where the soil mixed wall is exposed to the elements. Soils with high organic content are difficult to improve sufficiently and economically. Further, the strength of DMM is both time-dependent and variable with depth, as a function of the subsurface strata. The effect of time-dependency may require laboratory testing times up to a few months, although a 28-day strength is the usual strength index.

- It should be noted that dense sand and gravel or boulder layers cannot be penetrated by DMM equipment and may need costly excavation and removal prior to DMM construction, either by the dry or wet methods. Subsurface utilities and lifelines must be relocated in order for DMM to be performed. Both wet and dry rigs require substantial headroom.

- Although the quantity of waste is generally less than for some other technologies, DMM may produce between 30 – 100 percent of spoil soil volume, depending on project specifics, equipment, methods used, and in-situ water content. Disposal of the waste may be a significant cost item.

- At present, the lack of well-developed design and analysis models is a limitation to widespread acceptance. ***Further, the lack of standardized quality control testing and sampling makes design verification difficult and subjective***.

b. Dry Mixing, Lime, Cement, and Lime-Cement Columns (LC, CC, LCC)

Advantages

- In soft clay, LCC are often an economical alternative to conventional foundation methods (piles or drilled shafts), depending on such project factors as the size, weight, and flexibility of the structure, the depth and shear strength of the compressible strata, the risks and the consequences of a failure, and the effects of lowering the groundwater level.

- For road embankments and dikes constructed on soft clay, the lime or lime-cement column method can be an economical alternative to other ground improvement methods. The consolidation time using lime columns is reduced due to the increased permeability or stiffness of the columns, as shown in Figure 3. The LCC method can thus be an economical alternative to other methods such as lightweight fills, counterweight berms, excavation and replacement, and embankment piles.

- An advantage of the installation of LCC and CC by the dry mix method is the discharge of limited to no waste material to the ground surface, which eliminates potentially high disposal costs and offers lower mobilization costs compared to wet mixing. The use of cement columns as developed in Japan has not been demonstrated to date in the United States on transportation projects.

Disadvantages/Limitations

The full strength of the lime columns may not be mobilized when the pH value of the groundwater is very low (acidic) or the content of carbon dioxide (CO_2) is high. Low-strength mobilization should also be anticipated when mixing with non-reactive cohesive soils (clays lacking pozzolans). These soil conditions are not common.

The air-driven injection process may accumulate large quantities of air in the ground, which may cause heave to adjacent structures. This problem may be ameliorated by adding mixing paddles to the mixing tool, and/or substantially increasing mixing time, or by other means.[23]

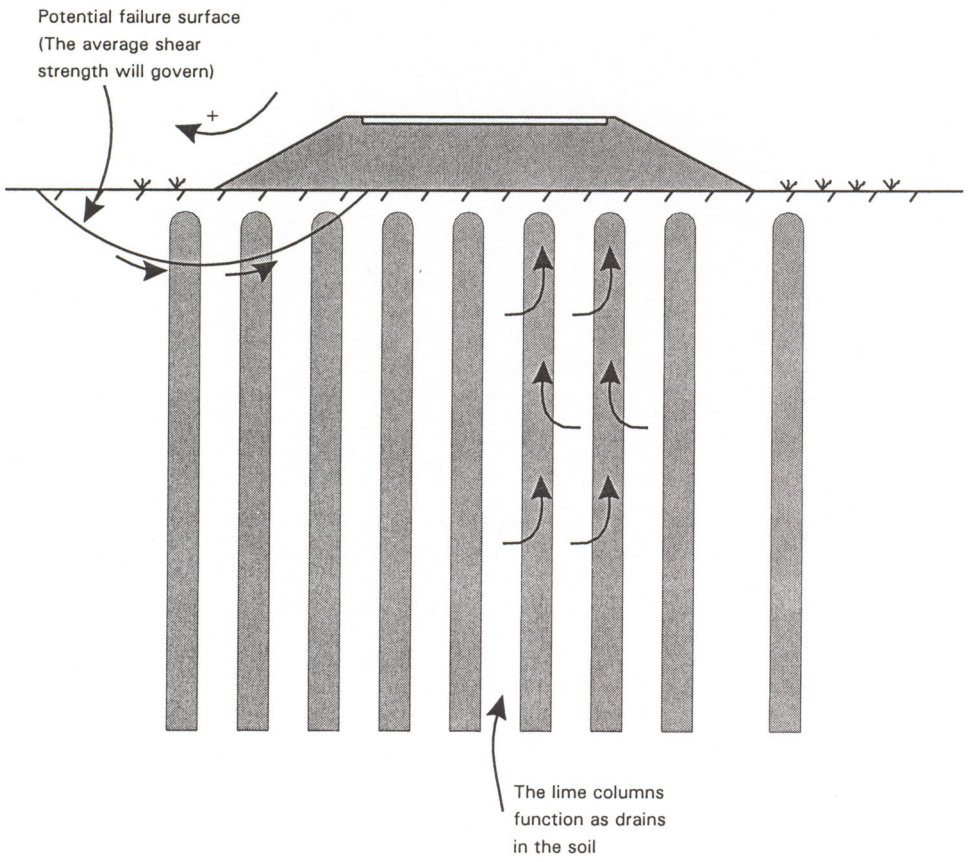

Figure 3. Lime Columns Installed for a Road Embankment.

- The creep strength of the lime columns and the shear strength of the stabilized soil is time dependent. Therefore, it may take several months to perform the necessary laboratory tests. For design, consider a reduced strength (28-day) to compensate, as creep strength is related to unconfined compressive strength. The average shear strength of the stabilized soil has to be at least three to five times the initial shear strength before the lime or lime-cement column method becomes economical.

- Relatively extensive field and laboratory investigations are normally required to develop design parameters. At present, the lack of well-developed design and analysis models is a limitation to widespread acceptance. ***Further, the lack of standardized quality control testing and sampling makes design verification difficult and subjective.***

2.3 FEASIBILITY EVALUATION

If solidified/stabilized soils are considered to be a viable engineering solution based on the soil profile, then a feasibility evaluation should be undertaken to determine the practicability of applying soil mixing.

Site Investigation

The site investigation should consider both surface and subsurface conditions. Both wet and dry mixing stabilization involve the use of very large equipment; therefore, restricted site access, inadequate working areas, or the presence of overhead utilities will generally preclude this technology. Underground utilities and obstructions must also be located, identified, and relocated. If relocation is not possible, then an alternative solution will be required in that location. With wet mixing techniques, a significant percentage of the volume of treated soil may be displaced, requiring adequate on-site storage prior to disposal off or on-site.

The design site investigation should include

- evaluation of soil type: governing soil type; existence of any type of obstruction; existence and percentage of organic matter
- in-situ water content
- engineering properties, such as strength, compressibility, plasticity, and classification index properties
- pH
- chemical and mineralogical properties to assess presence of pozzolanic materials, including soluble silica and alumina, which affects lime reactivity only
- groundwater levels

Feasibility Assessment

In general, a soil mixing solution may be most attractive where the site investigation has disclosed a soft to loose soil profile with no obstructions to depths no greater than 30 m (100 ft.). Further, there should be unrestricted overhead clearance, and a need for a relatively vibration-free technology to improve large volumes of soft soil. It should be understood that DMM methods cause a temporary loss of shear strength, which might adversely affect existing slopes with marginal stability or cause lateral ground movement to immediately adjacent structures.

In terms of achieved engineering properties for the improved soil, the literature indicates that with dry mixing, unconfined compressive strengths in the range of 0.2 – 2 MPa (29 – 290 psi), but more typically 0.6 MPa (87 psi), can be achieved.[1] With wet mixing techniques presently available in the United States (2003), the reported range is much wider, 0.2 – 10 MPa (29 – 1450 psi), but more typically in soft soils, in the range of 0.2 – 3 MPa (29 – 435 psi).[1]

Table 1 summarizes the range of anticipated engineering properties that can be achieved by soil mixing techniques.[1] Note that the wide range of reported improved properties is a function of the in-situ soils, reagent content, the equipment used for mixing the in-situ water content and organic content. Strength has a high degree of variability, but a minimum specified strength should be readily achievable.

It should be noted that certain soil chemistry factors significantly impede soil stabilization with cement. Soil chemistry limitations are summarized in Table 2.

For soils outside these limits, stabilization is possible at lower strength levels by the addition of certain stabilizers and special additives.

Table 1. Typical improved engineering characteristics.

Property	Typical Range
Unconfined Compressive Strength q_u	0.2 – 5.0 MPa (29 – 725 psi) granular soils 0.2 – 3.0 MPa (29 – 435 psi) cohesive soils
Permeability (k)	10^{-4} to 10^{-7} cm/s
Young's Modulus (E_{50}) (Secant Modulus at 50% q_u)	50 – 1000 q_u for lab samples (Japan) 50 – 150 q_u for lab samples (US) 100 – 300 q_u (typical range)
Tensile Strength (wet mix)	8 – 14 percent of q_u
Poisson's ratio	0.19 – 0.45 typically 0.26

Table 2. Favorable soil-chemistry factors.

Property	Favorable Soil Chemistry
pH	Should be greater than 5
Natural water content (w_c)	Should be less than 200% (dry method) and less than 60% (wet method)
Organic Content	Should be less than 6% (wet method)
Loss on Ignition	Should be less than 10%
Humus Content *	Should be less than 1.0%
Conductivity, electrical	Should be greater than 0.4 mS/cm

**Determined by loss on ignition after ashing for 1 hour at 800°C*

Preliminary Testing

Given the significant range of engineering properties that may be achieved by soil mixing techniques, a preliminary laboratory testing program to more narrowly determine reagent requirements and improvement levels may be considered even if a performance type of specification is used for construction. Such a program reduces the bidding risk to contractors and provides a reasonable starting point for the pre-production testing program.

The key physical and engineering properties of the composite material will depend on the type and percentage of the reagents utilized, the character of the in-situ soil, and the method and duration of mixing. In the design of the laboratory testing program, the intended use of the mixed soil must be considered. For example, the designer should determine if the mixed soil composite material is to be exposed to the effects of freeze-thaw cycles. Components of a typical laboratory testing program to determine strength and/or permeability are

- tests at multiple water/reagent (volume ratio) ratios. (wet mixes only)
- tests at multiple reagent content per unit volume soil
- tests at multiple mixing times
- tests with various additives (e.g., dispersants)
- all test results should be conducted as a function of time

These preliminary laboratory test results may more narrowly determine the potential improvement levels and provide significant advance information for the final design

program. ***Note, however, that very important variables associated with equipment mixing capabilities, such as rate of penetration and withdrawal, mixing energy, and vertical circulation of materials, cannot be modeled by the laboratory test program.*** Further note that for significant stabilization to occur, a minimum cement content of 5% must be used.

Environmental and Site Considerations

When deep soil stabilization is utilized for construction on environmental projects, the in-situ soil is used as part of the end product, thus reducing the expense and consequences of disposal by off-site hauling. For site improvement projects, the spoil from a wet mix program will be proportional under the best conditions to the amount of additives used for the mix. Quite frequently, the resulting waste contains a high percentage of cementitious material that can be used constructively on the site.

Dry mix methods are most advantageous where the in-situ silts or clays have very high moisture content and relatively low strengths are required for design. In these soils, the dry reagents will more easily mix with the in-situ soil to produce a uniform in-place mixture. Wet systems appear to have a broader application range, typically yielding higher strengths and significant spoil quantities. Optimum results are obtained where the in-situ water content of the clays and silts are in the 45 – 80 percent range.

The key issue to be resolved for site remediation applications is what type of chemical or other reagent should be injected and mixed into the contaminated soil, and in what quantities they should be injected. This is typically determined with bench scale testing. Actual soil samples are taken from a site, analyzed for pollutants and soil properties, and tested for pretreatment properties to be compared with post-treatment properties. Soil samples are then mixed with varying amounts of one or more reagents and tested to obtain post-treatment results.

2.4 AVAILABLE DEEP SOIL MIXING TECHNOLOGIES

Although both dry and wet methods have been used extensively overseas, they are still considered to be emerging technologies in the United States. A lack of generic design and construction guidelines, as well as the traditional difficulty of advancing a new technology have contributed greatly to the limited understanding and acceptance of these techniques to date.

In addition, the equipment used is quite diverse with often-proprietary features that can significantly influence the properties of the improved soil. Table 3 lists the operational soil mixing techniques presently available in the United States, describing key features, installation details, range of mix designs and reagents, and reported treated properties.[1] Given the fluid nature of the construction industry this list may not be complete or up to date.

Note that the volume ratio is an indication of the amount of spoil generated. The higher the ratio, the larger the amount of spoil.[1]

With respect to the effect of operational equipment features on the in-situ product, a review of subcommittee reports compiled by the Japanese Geotechnical Society Committee on Cement Stabilization Techniques suggests the following:[1]

- Higher strength and less variation is obtained by mixing with multiple shafts, and/or multiple paddles and restroking.
- Orientation of mixing blades can affect strength.
- Higher rotational speeds produce higher strength.
- Better mixing performance is obtained when injecting during penetration. This may not be always feasible.
- Reduction of penetration/withdrawal rate increases strength.

Given the above, the current practice has evolved with the following additional requirements:

- use of reagent content in the range of 150 – 300 kg/m^3
- mixing rotational speeds of 20 – 45 rpm
- penetration rates of about 1m/min
- water cement ratio of 1.0, but possibly in the range of 0.6 – 1.3

Much recent research and interest has been directed toward developing indicators of the potential efficiency of the mixing process that would produce a more homogeneous in-situ product of higher strength. It has been suggested by Japanese researchers that efficiency of a particular system can be established or expressed in terms of "the number of mixing per meter, T," which is related to certain operational and reagent injection characteristics as follows:

$$T = N\left(\frac{Rp}{Sp} \bullet \frac{Wi}{W} + \frac{Rw}{Sw}\right)$$

where N = total number of mixing blades
Sp, Sw = penetration and withdrawal speed (m/mm)

Rp, Rw = rotational speed of blades during penetration and withdrawal (rpm)

Wi = stabilizer injection on penetration (kg/m^3)

W = total amount of stabilizer (kg/m^3)

The research suggests that good quality product with greater uniformity is obtained when T, the number of mixing, is greater than 350 for clays and between 400 and 450 in peaty soils.

Table 3. Details of operational soil mixing techniques.[1]

Classification **Name** **Company** **Geography**	WRS DSM GeoCon N. America	WRS SMW Raito Inc. S.E. Asia, U.S.
General Description of Method	Adjacent discontinuous augers rotate in alternate directions. Most of grout injected mainly on downstroke. Water and air not typically used during penetration.	Adjacent discontinuous augers rotate in alternate directions. Water, air, or grout used on downstroke, and grout on upstroke.
Special Features/Patented Aspects	Lower 3m usually double stroked. Strong QA/QC by electronic methods.	Special electric head patented Double stroking "oscillation" common especially in cohesive soils. Discontinuous auger flights are positioned at intervals to reduce torque requirements.
Details of Installation Shafts Diameter Depth RPM Productivity/output	 1 - 6, usually 4 0.8 - 1.0m, usually 0.9m 45m 15 - 25 0.6m/min penetration 2 m/min withdrawal/mixing, 100 - 150m^3 per shift	 2 - 5, usually 3 0.55 - 1.5m, usually 0.9m 60m claimed, 35m practical 15 - 20 0.5m/min penetration 2m/min withdrawal/mixing 100 - 200 m^3 per shift
Mix Design *(depends on soil type and strength requirements)* Materials w/c ratio Cement ratio (kg_{cement}/m^3_{soil}) Volume ratio (Vol_{grout}:Vol_{soil})	 Cement grout $\pm$ bentonite and other additives w/c = 1.2 - 1.75 120 - 400kg/m^3 soil 15 - 40%	 Cement grout $\pm$ bentonite and other additives w/c = 1.3 - 2.5 250 - 750 kg/m^3 soil 50 - 100%
Reported Treated Soil Properties U.C.S.* Permeability, k Modulus, E	 0.3 - 7 MPa (clay strengths approx. 40% of sands) 1 x 10^{-7} - 1 x 10^{-9} m/s 300 - 1000 x U.C.S.	 0.3 - 1.3 MPa (clay) 1.4 - 4.2 MPa (sands) 1 x 10^{-7} - 1 x 10^{-10} m/s 500 - 1350 x U.C.S.
Specific Relative Advantages and Disadvantages	Economic, proven systems. Mixing efficiency can be poor in stiff, cohesive soils. Can generate large spoil volumes.	
Notes	Developed in late 1980s.	Developed in 1972; first used 1976 in Japan, 1986 in U.S.

* U.C.S. - Unconfined compressive strength

Table 3 (cont'd). Details of operational soil mixing techniques.[1]

Classification **Name** **Company** **Geography**	WRE SCC SCC Technology SCC (U.S.): Tenox (Japan)	WRE HBM Hayward Baker Inc.: A Keller Co. (U.S. and worldwide)
General Description of Method	Grout is injected during penetration. A non-rotated "share blade" is located above tip. At target depth, 1 minute of additional injection plus oscillation for 1.5 - 3m. Withdrawal with counter rotation.	Kelley attached shaft rotated by top rotary drive head. Grout injected during penetration, followed by 5 minutes mixing and oscillation at full depth, and rapid extraction with injection of "backfill grout" only (1 - 5% total).
Special Features/Patented Aspects	Very thorough mixing via "share blade" action, which is patented.	Method proprietary to Keller.
Details of Installation Shafts Diameter Depth RPM Productivity/output	 Single with 3 rotated mixing blades plus "share blade" 0.6 - 1.5m; 1.2m double shafts 20m max 30 - 60 1 m/min in and out 400m of piles/shift	 Single with 2 or 3 pairs of paddles above drill bit 0.5 - 3.5m, typically 2.1m to 2.4m 20m max 20 - 40 (penetration): higher upon withdrawal 0.3 - 0.5 m/min (penetration): faster upon withdrawal. In excess of 500 m^3/shift
Mix Design *(depends on soil type and strength requirements)* Materials w/c ratio Cement ratio (kg_{cement}/m^3_{soil}) Volume ratio (Vol_{grout}:Vol_{soil})	 Typically cement, but others e.g., ash, bentonite, possible. 0.6 (clays) to 1.2 (sands) 100 - 450 kg/m^3 cement 30 - 35%	 Varied in response to soil type and needs. 0.6 - 1.5 150 kg/m^3 cement 15 - 30%
Reported Treated Soil Properties U.C.S.* Permeability, k Modulus, E	 3.5 - 7 MPa (sands) 1.3 - 7 MPa (cohesives) 10^{-8} m/s 180 U.C.S.	 3.5 - 10 MPa (sands) 0.2 - 1.4 MPa (clays) 10^{-10} m/s
Specific Relative Advantages and Disadvantages	Low spoil with minimal cement loss claimed, due to low w/c and minimized injected volume.	Good mixing, moderate penetration: low spoils volume. Dry binder may be used.
Notes	Used since 1979 in Japan and 1993 in U.S.	Developed in 1990.

* U.C.S. - Unconfined compressive strength.

Table 3 (cont'd). Details of operational soil mixing techniques.[1]

Classification **Name** **Company** **Geography**	WJE GeoJet Condon Johnson Associates Western U.S.	DRE Lime-Cement Columns Underpinning & Foundation Co. Uses Stabilator technology, U.S.
General Description of Method	Grout is jetted via ports on a pair of wings during penetration. The wings cut the soil and the jetted grout blends it.	Soil is penetrated while injecting compressed air above the horizontal blades. Dry materials are injected during withdrawal via compressed air and reverse rotation.
Special Features/Patented Aspects	Combination of mechanical and hydraulic cutting/mixing gives high-quality mixing and fast penetration. Licensed by CJA.	Very low spoil. High productivity. Efficient mixing. Patents are held by the contractors. Strong reliance on computer control.
Details of Installation Shafts Diameter Depth RPM Productivity/output	 1 shaft with pair of wings. 0.6 - 1.2m 45m max (25m typical) 150 - 200 (recent developments focusing on slower rpm) 2 - 12 m/min (penetration) 15 m/min (withdrawal) 150m of piles/hr possible	 Single shaft, various types of cutting/mixing blades. 0.5 - 1.2 m, typically 0.6, 0.8m 25m max 5 - 15m/min (penetration) 2.5 - 5m/min (withdrawal) 140 - 200 rev/mm (withdrawal) 350 - 1000 lin m/shaft
Mix Design *(depends on soil type and strength requirements)* Materials w/c ratio Cement ratio (kg_{cement}/m^3_{soil}) Volume ratio (Vol_{grout}:Vol_{soil})	 Cement, additives 0.5 - 1.5 (1.0 typical) 150 - 300 kg/m^3 20 - 40%	 Cement and lime in various percentages (typically 50:50 or 75:25) 23 - 28 kg/m (600mm diameter); 40 kg/m (800mm diameter) i.e., 80 - 150 kg/m^3
Reported Treated Soil Properties U.C.S.*	0.7 - 5.5 MPa (S.F. Bay mud) 4.8 - 10.3 MPa (other clays0	Varies, but typically 0.2 - 0.3 MPa (0.2 - 2 MPa, possible)
Specific Relative Advantages and Disadvantages	Computer control of penetration parameters excellent. High strength. Low spoil volumes. Excellent mixing.	Very low spoil. Computer controls of penetration parameters and reagent usage.
Notes	Developed in early 1990s.	Developed by Swedish industry and government, with first commercial applications in mid 1970s.

** U.C.S. - Unconfined compressive strength.*

Table 3 (cont'd). Details of operational soil mixing techniques.[1]

Classification **Name** **Company** **Geography**	WJE Hydramech Geo-Con, Inc. U.S.	WJE Spread Wing (SWING) Raito Inc. Japan, U.S.
General Description of Method	Drill with water/bentonite or other drill fluid to bottom of hole. No compressed air used. At bottom, start low-pressure mechanical mixing through shaft. Cycle three times through bottom zone. Multiple high-pressure jets started at same time. (350 - 450 MPa)	With blade retracted, 0.6-m diameter pilot hole is rotary drilled to bottom of zone to be treated. Blade expanded and zone is treated with rotary mixing to 2-m diameter and air jetting to 3.6 m diameter.
Special Features/Patented Aspects	3-mm-diameter "hydra" nozzles on outer edges of mixing tool. Mechanical mixing occurs in center of columns, chunks of soil forced to perimeter where disaggregation occurs by jets.	Retractable mixing blade allows treatment of specific depths to large diameter. Concentric mechanically mixed and jet mixed zones are produced. Patented. Trade association.
Details of Installation Shafts Diameter Depth RPM Productivity/output	 1 1.2-m paddles on 0.9-m auger; mixed column up to 2-m diameter, depending on jet effectiveness. 20+ m 10 - 20 Up to 500 m^3/shift	 1 0.6-m pilot hole, 2.0-m (mechanical) to 3.6-m (jetted) column. 40m N/A 0.03 - 0.1m/min penetration
Mix Design *(depends on soil type and strength requirements)* Materials w/c ratio Cement ratio (kg_{cement}/m^3_{soil}) Volume ratio (Vol_{grout}:Vol_{soil})	 Cement 1.0 - 1.5 100 - 250 kg/m^3 10 - 15% by weight of soil.	 Cement grout N/A 450 kg/m^3 N/A
Reported Treated Soil Properties U.C.S.* Permeability, k Modulus, E	Up to 10 MPa Up to 1 x 10^{-9} m/s 100 – 300 x U.C.S.	0.4 - 4.4 MPa (mechanically mixed zone); 1.5 MPa (sandy), 1.2 MPa (cohesive) (jet-mixed zone) 1x10^{-8} m/s 150 x U.C.S. (mechanically mixed zone); 100 x U.C.S. (jet-mixed zone)
Specific Relative Advantages and Disadvantages	No air used. Very uniform mixing. Control over diameters provided at any depth. Several times cheaper than jet grouting. Mixing can be performed within specific horizons, i.e., plugs can be formed instead of full columns.	Variable column size generated by varying pressures; retractable/expandable blade, jet mixing allows good contact with adjacent underground structures in difficult access areas.
Notes	Field-tested at Texas A & M. Fully operational since 1998.	SWING Association with 17 members established in late 1980s in Japan.

* *U.C.S. - Unconfined compressive strength.*

Table 3 (cont'd). Details of operational soil mixing techniques.[1]

Classification **Name** **Company** **Geography**	WRS DMW / DMI Schnabel Foundation Co. U.S.	WRE MecTool® Millgard Corporation U.S. and U.K.
General Description of Method	Cutting tools and paddles located on adjacent shafts. Shafts rotate in alternate directions. Most grout is injected on the downstroke. Air used sometimes in clays during downstroke.	For cohesive soils, grout is placed in pre-drilled hole in center of each element, and soil in the annulus of the tool is then blended with mixing tool. End mixing with grout injected through hollow kelly bar.
Special Features/Patented Aspects	Automatic grout plant controlled by soil mixing equipment operator. Continuous electronic monitoring of soil mixing operation.	MecTool. Also Aqua MecTool, which is an isolation mechanism that encloses submerged mixing tool in remediation zone providing protection against secondary contamination.
Details of Installation Shafts Diameter Depth RPM Productivity/output	 4 0.9m cutting tool 35m max 12 - 25 100 - 300 m^3/8-h shift 0.6 m/min in; 2m/min out	 1 1.2 - 4.2m max 25m max (typically less than 6m) N/A 0.6 m/min.
Mix Design *(depends on soil type and strength requirements)* Materials w/c ratio Cement ratio (k_{cement}/m^3_{soil}) Volume ratio (Vol_{grout}:Vol_{soil})	 Cement grout mainly, and bentonite sometimes. 1.0 - 2.0 200 - 750 kg/m^3 30 - 100% by weight	 Cement grouts, including PFA and other materials ± proprietary additive to breakdown "plastic seals thereby enabling through-the-tool delivery" N/A N/A 20 - 35% estimated range
Reported Treated Soil Properties U.C.S.* Permeability, k Modulus, E	 Sand - 1 - 3 MPa Clay – 0.3 - 1.4 MPa 1 x 10^{-7} to 10^{-10}	 0.8 - 2.5 MPa 1 x 10^{-8} to 1 x 10^{-9} m/s N/A
Specific Relative Advantages and Disadvantages	Economically proven system. Larger spoil in stiff clays.	Excellent control of grout and spoil quantity.
Notes	Developed in late 1980s.	Mainly environmental applications to date.

** U.C.S. - Unconfined compressive strength.*

Table 3 (cont'd). Details of operational soil mixing techniques.[1]

Classification **Name** **Company** **Geography**	WRE SSM Geo-Con, Inc. U.S.	WRS Multimix (Trevimix) Trevisani Italy, U.S.
General Description of Method	Single large-diameter auger on hanging leads or fixed rotary table, is rotated by bottom rotary table and slurry or dry binder is injected. Auger rotation and injection continue to bottom of treated zone. Auger rotation during withdrawal usually without injection.	Multiple cable-suspended augers rotate in opposite directions. Grout injected during penetration. Prestroked with water in clays. Auger rotation reversed during withdrawal. Mixing occurs over 8 – 10 m length of shaft.
Special Features/Patented Aspects	Single large-diameter auger; cycling up and down is common to improve mixing efficiency.	Pre-drilling with water $\pm$ additives in very resistant soils. Process is patented by TREVI. Developed especially for cohesionless soils of low/medium density, and weak clays.
Details of Installation Shafts Diameter Depth RPM Productivity/output	 1 1 - 4m 12 m 15 500 - 1500 m^3 per shift	 1 - 3, typically 3. Configuration varies with soil. 0.55 - 0.8 m at 0.4 to 0.6-m spacings 25m 12 - 30 0.35 - 1.1 m/min penetration (typically 0.5) 0.48 - 2 m/min withdrawal
Mix Design *(depends on soil type & strength requirements)* Materials w/c ratio Cement ratio (kg_{cement}/m^3_{soil}) Volume ratio (Vol_{grout}:Vol_{soil})	 Cement grout, bentonite, flyash, lime, and other additives for contaminant immobilization 1 - 1.75 200 - 400 kg/m^3 12 - 20%	 Cement grout mainly, plus bentonite in sands; additives common, even in predrilling phase Typically low, i.e., 0.6 - 1.0 (especially in cohesive soils) 200 - 250 kg/m^3 typical (80 - 450 kg/m^3 range) 15 - 40%
Reported Treated Soil Properties U.C.S.* Permeability, k Modulus, E	 3.5 - 10 MPa in granular soils 0.6 - 1.2 MPa common in high-water-content sludges 1 x 10^{-10} m/s possible 100 to 300 x U.C.S.	 0.5 - 5 MPa (sands); 0.2 - 1 MPa (silts, clays); up to 20 MPa in very hard soils <1 x 10^{-8} m/s N/A
Specific Relative Advantages and Disadvantages	Can treat wide variety of contaminants, including creosote, tar, organics, petroleum, etc.	Goals are to minimize spoils (10 - 20%) and presence of unmixed zones within and between panels.
Notes	Mainly used for environmental applications to date, but increasing use in geotechnical field.	Developed jointly in 1991 by TREVI and Rodio.

** U.C.S. - Unconfined compressive strength.*

Table 3 (cont'd). Details of operational soil mixing techniques.[1]

Classification **Name** **Company** **Geography**	DRE Trevimix TREVI-ICOS Italy, U.S., Far East
General Description of Method	Soil structure disintegrated during penetration with air. Augers are then counter-rotated on withdrawal and dry materials are injected via compressed air through nozzles on shaft below mixing paddles. Binder can also be added during penetration.
Special Features/Patented Aspects	Use of "protection bell" at surface to minimize loss of vented dry binder. System is patented by Trevi and also used under license by Rodio. Needs soil with moisture content of 60 - 145+%, given relatively high cement factor and diameter.
Details of Installation Shafts Diameter Depth RPM Productivity/output	 1 - 2 (more common). Separated by fixed (but variable) distance of 1.5 - 3.5 m. 0.8 - 1.0 m (most common) 30m 10 - 40 0.4m/min penetration 0.6m/min withdrawal 39m/8-h shift
Mix Design *(depends on soil type and strength requirements)* Materials w/c ratio Cement ratio (kg_{cement}/m^3_{soil}) Volume ratio (Vol_{grout}:Vol_{soil})	 Dry cement (most common), lime, max. grain size 5mm N/A 150 - 300 kg/m^3 N/A
Reported Treated Soil Properties U.C.S.* Permeability, k Modulus, E	 1.8 - 4.2 MPa (avg. 2.5 MPa) N/A 1 to 2.66 x 10^3 MPa (clays) 3.1 x 10^3 MPa (sandy soils)
Specific Relative Advantages and Disadvantages	No spoil, uniform mixing, automatic control of binder quantity. System allows for "possibility of injecting water during penetration."
Notes	Developed by TREVI in Italy in late 1980s. Trevi-ICOS, U.S. licensee, in Boston, MA.

* *U.C.S. - Unconfined compressive strength.*

CHAPTER 3
CONSTRUCTION AND MATERIALS

This chapter focuses on construction methods and equipment and the types of materials used in each process.

3.1 CONSTRUCTION METHODS AND CONSTRUCTION EQUIPMENT

There are many contractors currently involved in deep soil mixing, as shown in Table 3, resulting in variations in equipment, procedure, reagents, and additives used. The following discussion is in generic terms to provide an overview of typical construction methods and equipment.

a. Wet Deep Mixing Methods

DMM is performed by individual or multiple-stem mix tools, either barge-mounted for off-shore work, as shown in the schematic in Figure 4, or based on tracked cranes for work on land, as shown in Figure 5. Multi-auger equipment is shown in Figure 6.

For offshore marine works, the DMM machine usually has more than two mixing tools, and can have up to eight tools if required. A typical off-shore operation would be accomplished with a complete, barge-mounted working unit composed of a silo for the reagent (usually cement), a grout mixing plant, a support tower for the soil mixing equipment, and a control center for control of the soil mixing operation and barge orientation.[3] Machines are available that have the capability of constructing columns with cross-sectional areas ranging from 1.5 – 9.5 m^2 (16 – 100 ft^2) with the maximum depth of treatment being up to 70 m (230 ft.) from the water surface.[3]

For on-shore operations a conventional, track-mounted crane typically supports the soil mixing equipment. The capacity of the base machine is a limiting factor on the number of mix tools that can be safely accommodated, and usually a maximum of four tools can be mounted on the support mast. Also, since the length of the leaders supporting the mix tools is limited, maximum improvement for land-based operations is less than for off-shore operations, typically 30 m (100 ft.) as compared to up to 70 m (230 ft.).[1] The mixing plant generally consists of a slurry mixer, slurry agitator, batching system, slurry pumps, and a computer for mixing and slurry control.

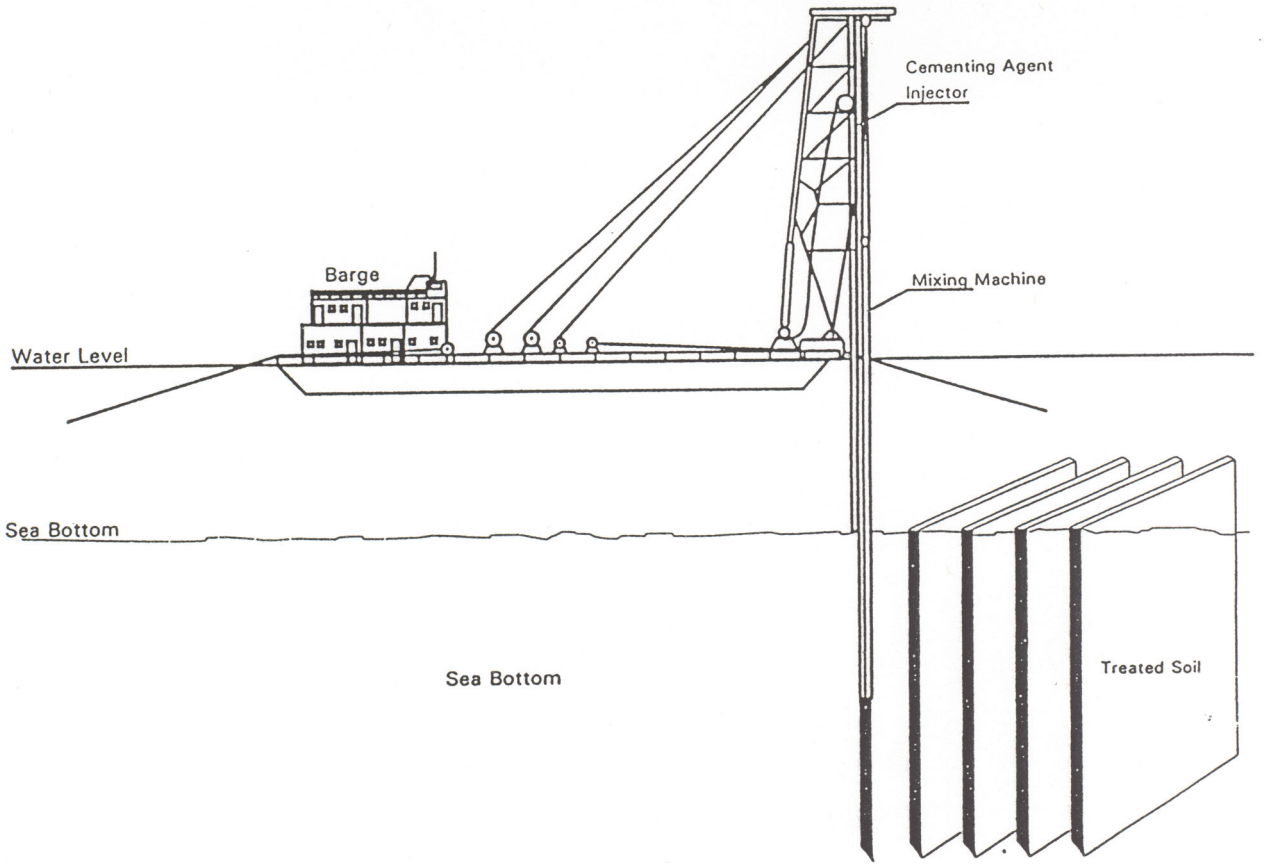

Figure 4. Schematic of Barge-Mounted Deep Soil Mixing Rig.

Figure 5. Track-Mounted Equipment.

Figure 6. Multi-Auger Mixing Equipment.

An automated batching system should be used that measures the water, cement, and other additives by weight to produce more consistent slurry. The desired weight of each slurry component can be preset. Mix design changes are made by adjusting the component at the control panel. A separate positive displacement pump supplies the slurry to each of the injection augers for accurate control of slurry flow.

The soil mixing process is similar for offshore and land-based operations. The wall or block formation is constructed using overlapping mixing tools guided by leads. The tools are positioned on the leads so as to overlap one another to form the continuous overlapping columns as the work proceeds. The mixing tools are typically driven by a top-drive gear box and motor that provides enough torque to maintain a continuous installation of the soil-mixed columns. The lead is supported at separate points for accurate control of vertical alignment, which is critical to prevent unmixed zones between column sets and to maintain continuity of the soil-cement wall. As the mixing tools are advanced into the soil, grout is pumped from the mixing plant through the hollow stems of the mixing tools and injected into the soil at the tip of the tool. The mix tool flights and mixing blades on the stems blend the soil with the grout in a continuous mixing fashion. When the design depth is reached, the mixing tools are withdrawn. The mixing process is repeated during the withdrawal stage. Upon completion of this process, a "panel" of overlapped soil-cement columns is left in place.[9]

The design of the mix tool flights and mixing paddles on each stem varies with soil type, and the mix tool design is frequently tailored to meet specific project conditions. Basic mix tool designs are in use for mixing cohesive soils, sand, and gravel. The flights and mixing paddles on adjoining stems are staggered at different elevations to produce the overlapped soil-cement columns after mixing. Joint bands are located periodically on the mix tool stems to maintain the space between adjoining stems, and to fix the mix tool as a rigid body to accurately produce one overlapped column panel with a single stroke. Drill heads, the portion of the auger tip that provides the initial drilling capability, are also designed to address different soil conditions and can be easily changed to meet varying conditions during the drilling process.

Although all of the presently available equipment in the United States have features to penetrate, inject reagent, and mix the in-situ soils, the results obtained are not generally homogeneous. **The soil mixed product is not a totally homogeneous mixture, and therefore, engineering properties can vary significantly within each column as a function of the penetration rate, mixing RPM, extractor rate, method of injection, reagent quantity, and in-situ natural moisture and organic content of the penetrated soils.**

A field pre-construction program prior to full-scale construction is required in which the effect of the installation method variables need to be quantified and procedures for full-scale construction adopted. These procedures should form the basis for construction control, as they would be most likely to produce the required and desired engineering characteristics.

It should be understood that soil mixing is not designed to dissolve the in-situ soil before blending it with the reagent powder or slurry. The cutting heads and mixing paddles are designed to break up the in-situ soil and mix it. For cohesionless soils with minor fines, sand and gravel particles become dispersed aggregates of the soil-cement mixture. When the fines content and plasticity index of the soil increase, the size/frequency of soil lumps increases. For highly plastic clays, part of the clay cannot be broken down during the soil mixing process and remains as lumps inside the soil-cement mixture. As long as the lumps are well dispersed inside the soil-cement, the performance of the soil-cement should be satisfactory. Therefore, the uniformity of soil-cement should be evaluated from large-scale viewpoint.

Figure 7 illustrates various mixing tools and blade orientations.

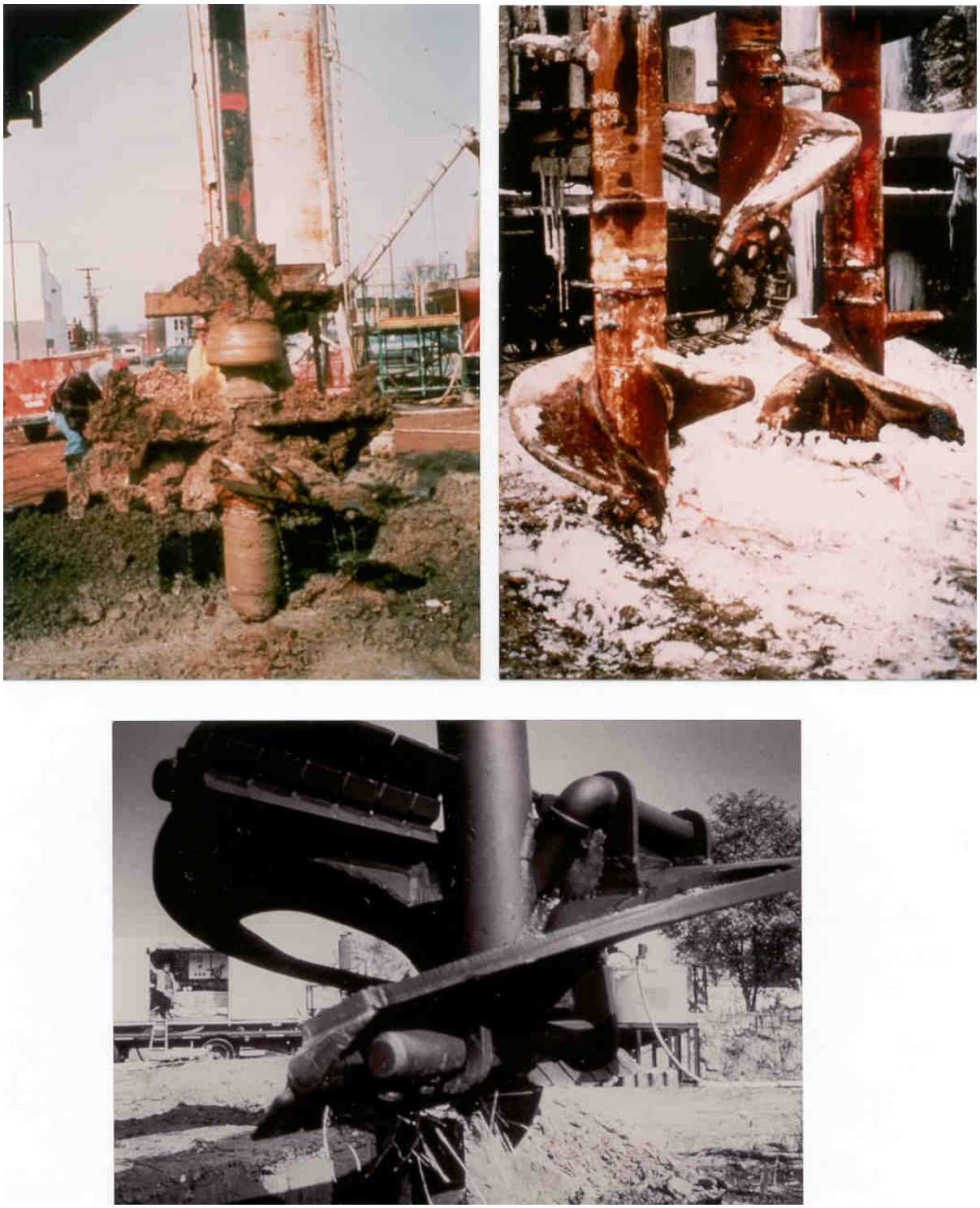

Figure 7. Various Mixing Tools and Blade Orientations.

Shallow soil mixing can be accomplished with a single, large diameter (from 1.0 – 3.7 m (3.3 – 12 ft.)) mix tool. The tool serves as both a boring tool and a mixing tool, as shown in the bottom of Figure 7. The mix tool is supported by a hollow stem kelly bar and reagent delivery system. The single auger system is typically limited to 15 m (50 ft.) in depth because of torque limitations.

The equipment is supported by a large track-mounted crane. A shroud can be utilized over the boring/mixing auger at the surface to prevent any discharge of air-polluting contaminants or reagent dust (Figure 8). Provided with an on-site mixing/pumping plant, a single rig has the capability of mixing over 300 m^3 (400 yd^3) of soil per 8-hour shift.

Figure 8. Shrouded Boring/Mixing Tool.

b. Dry Deep Mixing Method

Construction is typically accomplished with a single mix tool mounted on a modified crane or front-wheel loader. The binders are stored in containers located behind the loader on the rig or in a separate carrier. The carrier is connected to the installer by electrical cables and the hose for binders. The operator maneuvers the barrier from the installer. A self-contained unit is shown in Figure 9.

To construct lime-cement columns, the soil is mixed in-situ with the lime-cement mixture by the mix tool, usually described as being shaped like a giant dough mixer, as shown in Figure 10. The mixing tool is first rotated into the soil down to the depth that corresponds to the required length of the columns. The maximum depth is at present 30 m (100 ft.). The tool is then withdrawn as the additive lime-cement is forced down into the soil by compressed air pushing through holes located just above the horizontal blades of the mixing tool. Since the blades are inclined, the stabilized soil will be compacted during the withdrawal of the tool. The resulting diameter of the columns is 0.6 – 1.0 m (2.0 – 3.3 ft.).

Note that dry mixing methods using lime or cement only have been used in Europe and Asia.

Figure 9. Self Contained Unit, Drill and Binder Storage.

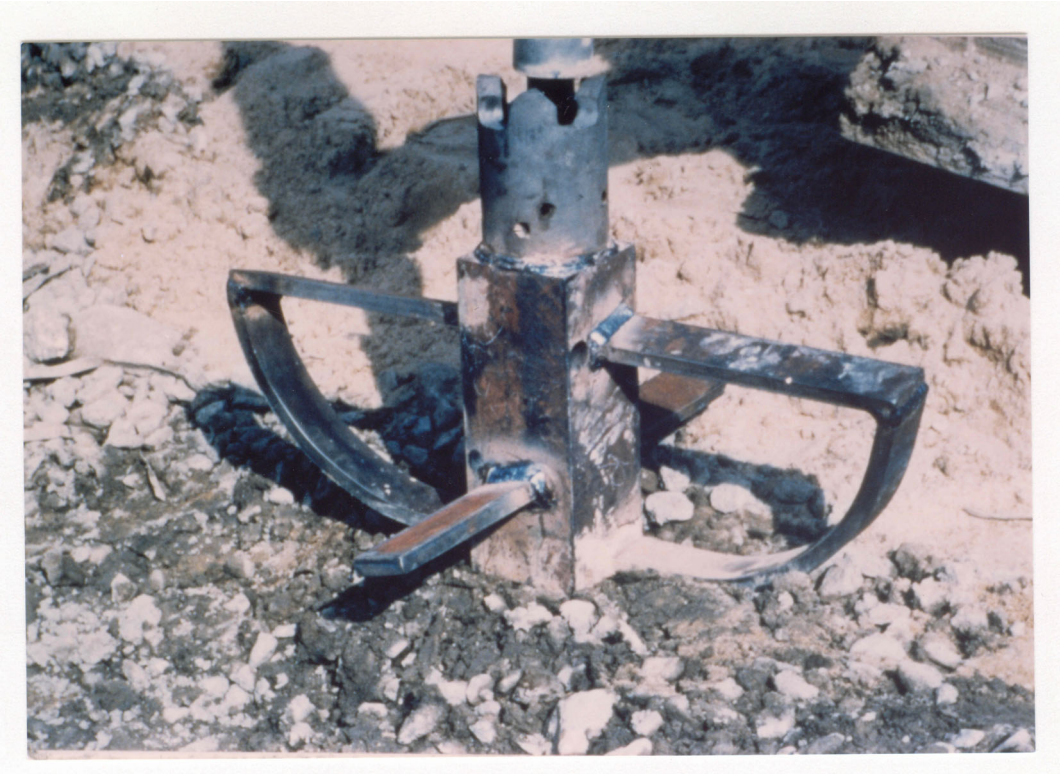

Figure 10. Mixing Tool for Lime-Cement.

3.2 MATERIALS

Large-scale wet DMM was originally developed with cement as the primary stabilizing agent and dry DMM technology using unslaked lime. At present, with dry technology, lime-cement is used most of the time with the lime proportion varying from 15 – 40 percent. For both types of deep soil stabilization, a range of other additives may also be included, depending on project-specific subsurface conditions.

a. Wet Mixing

Ordinary Portland cement is typically used in the DMM process. The cement is typically introduced as slurry rather than in dry form, since it results in a more uniformly mixed soil-cement product, which increases the strength and uniformity of the treated soil. However, because soil acidity can affect the hydration reaction in organic soils, specially blended improved cements should be used where organics are present. Bentonite and other cement additives are used to achieve specific design purposes, such as decreased permeability, greater workability, or early set.

The construction process uses the in-situ soil as aggregate and the cement-grout as binder and hardening agent to form the composite soil-cement product. Therefore, the engineering properties of soil-cement are influenced by

- the physical and chemical properties of the in-situ soils, in-situ strength, mineralogy, organic content, pH, and water content
- reagent content, type, quality, mixing water, and additives
- the degree of mixing, penetration, and withdrawal speed.
- auger and mixing paddle configurations
- curing conditions, temperature, time

The strength of the in-situ soil-cement is influenced by the variables outlined above. Therefore, a wide range of field strengths should be anticipated. Similar to concrete, the unconfined compressive strength of soil-cement is used as the reference strength value for design and quality control.

b. Dry Mixing

Lime Columns

Lime column construction uses unslaked lime. A rapid hydration reaction occurs when lime is added to wet soils, reducing the water content and thereby increasing the shear strength. Concurrently, a slower increase in shear strength is attributed to pozzolanic reactions when the lime reacts with silicates and aluminates in the clay.[(2)]

Along with the unslaked lime, other additives may be utilized. The most heavily used additive is gypsum, which may be combined (usually comprising about 33 percent of the mixture) with the unslaked lime to stabilize organic soils possessing water content of up to 120 percent. One of the advantages of this mixture is that gypsum also tends to speed up the chemical reactions. It has been noted that after 10 to 100 days, the shear strength of the stabilized soil can be two to four times greater than when unslaked lime is used alone.

Lime-Cement Columns

Presently, lime-cement mixtures are used almost exclusively, worldwide. The ratio of cement to lime varies, depending on application and target engineering properties, but is often at a 5 to 1 ratio, with the lower percentage being lime.

Other additives, such as fly ash, can stabilize organic soils where the content of water is greater than 100 percent.

The ground temperature of the site can have dramatic effects on chemical reaction rates and shear strength improvement. These rates are very low when the ground temperature is below 4°C. The surface of the site should be covered with straw, rockwool, etc., if the potential for ground freezing exists. Note that ground temperatures below an upper crust are controlled by groundwater temperatures varying between 10^{o} – 15^{o} C. Further, the heat of hydration of both cement and quicklime tend to raise ground temperatures.

When the ground temperature is high, the chemical reaction is accelerated. This is important if the project requires certain property changes within a set time period. If the columns are spaced 2 m (6.6 ft.) or less, the stabilized soil temperature may be raised to 30 – 50°C after three or four days.

Cement Columns (CC)

Dry mixing systems using only cement are available and widely used in Asia for soil profiles with high water contents (typically greater than 70 percent). The final product is similar to that produced by wet systems.

c. Additives

As originally conceived, lime columns used unslaked lime as the main additive. Gypsum was added to lime to stabilize organic soils or soils with high water content.

Presently, lime-cement is used almost exclusively at varying proportions to provide the target engineering properties. Site-specific bench scale testing is required to develop the optimum mix. The cement increases strength and stiffness, and reduces permeability.

In addition to the cement, other additives may be added depending on the soils to be treated. These include flyash, blast furnace slag, or gypsum. Currently in Japan, flyash is being used at the rate of 2/3 flyash to1/3 cement because of the need to dispose of large amounts of flyash. However, due to the variables in coal sources, the use of flyash requires a greater amount of pre-construction experimentation to determine the specific effects on soil-cement mixing, set times, and final strength. Technical use for fly ash or certain slags is warranted when the clay minerals do not contain adequate pozzolans to induce the pozzolanic reaction.

The activity of the in-situ soil and pozzolans may be evaluated by measurements of conductivity. The following ranges of conductivity may be used as indicators:

Conductivity in ms/cm	Pozzolanic Activity
< 0.4	non-pozzolan
> 0.4 <1.2	ordinary
> 1.2	good

Typically, high soil sulfide and/or sulfate contents and/or high organic contents (therefore, low pH) inhibit strength development, whereas chloride ions enhance pozzolanic activity.

Fly ash also acts as a retardant by reducing the heat of hydration. Typically, fly ash mixtures result in low unconfined compression strengths in the range of less than 0.5 MPa (72 psi).

Clay in the form of bentonite (less than 10% by weight of cement) can be added to produce a more impervious stabilized soil and to improve flowability and/or prevent bleeding. More typically, proprietary admixtures are added to the soil cement slurry to prevent bleeding and/or to delay setting. Other additives have been used for special purposes, as follows:

- Kiln dust alone or together is mixed with other reagents for treating soils with heavy metals and sludges.

- Dispersant additives are used to provide enhanced fluidity for lower water/cement ratios or retarders. Dispersants have been used to improve drill penetration rates and enhance mixing efficiency in stiffer cohesive soils.

- Proprietary mixtures, such as silicates, thermoplastics, and polymers, are used in conjunction with cement for the treatment of organic contaminants.

CHAPTER 4
FUNDAMENTALS OF DESIGN

The vast majority of soil mixing projects to date have been accomplished in Japan and Scandinavia, respectively. In the United States a limited number of transportation related projects have been undertaken, the largest in connection with the Central Artery construction in Boston, MA. Experience with lime columns is limited to a single test program, and with lime-cement on a project at I-15, Utah. Given the emerging nature of these techniques in the United States, design discussion in this chapter will draw extensively on the design parameters and methodology developed by Japanese and Scandinavian engineers, as modified by current United States experience.

This technical summary on soil mixing is intended to provide an introduction to the design techniques currently in use. ***It is not intended to be a comprehensive design and construction reference***. Accordingly, the basis of design, mix proportions and design methods covered in this chapter are addressed in general terms only and should not be used directly in the preparation of project-specific documents.

The user should review current literature and contact specialty contractors for current information and recommendations.

4.1 DESIGN BASIS

The deep soil mixing technique with either dry or wet reagents, can be applied to problems of excavation support, settlement reduction, global shear strength increase, groundwater control, liquefaction mitigation, and mass stabilization for structural support.

For each of these applications, the engineering properties and volumes of soil to be stabilized are site and project specific. Typical patterns of treated volumes are shown in Figure 11, and equipment demonstrating a block-type installation is shown in Figure 12.

Typically, the engineering properties that must be developed by soil mixing to implement design are tensile, compressive and shear strength, permeability, and modulus of elasticity.

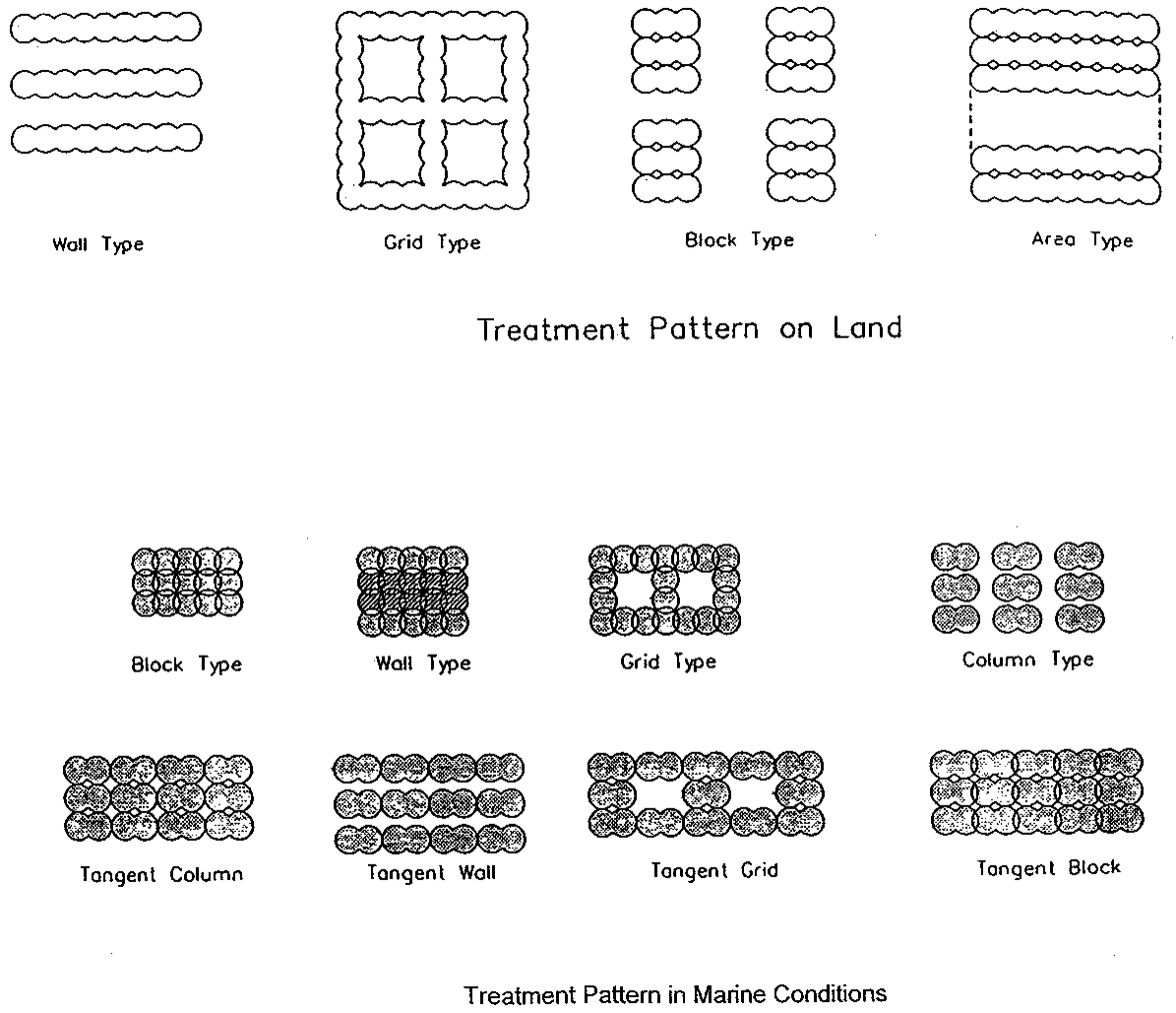

Figure 11. Basic Deep Mixing Treatment Patterns.[1]

Figure 12. Equipment Installing a Block-Type Pattern.

These targeted required properties for each site-specific use are developed from project structural or performance requirements. For excavation support applications, minimum compressive and tensile strength to achieve structural integrity of the facing are targeted, as is modulus, in order to meet deflection performance criteria. For groundwater control, a minimum permeability is targeted for mass stabilization minimum compressive strength and for settlement reduction/shear strength increase, compressive strength. Design issues are further discussed with respect to applications.

4.2 DESIGN CONCEPTS BASED ON APPLICATIONS

a. Excavation Support and Groundwater Control

A soil mix wall ability to form an impermeable barrier and to have the potential to be reinforced with soldier piles make this system a candidate for excavation support walls. For deep excavations, tieback anchors are installed in horizontal rows. In the United States, for highway applications, a significant future use is expected to be for excavation support and groundwater control using wet mix technology.

The design of walls with a soil mix face is similar to the design of any excavation support wall that is considered flexible. The pattern used is a continuous overlapping single or double row pattern.

The cross-section design of soil-cement facing for excavation support consists of designing the vertical members (H-piles, etc.) to resist bending moments, shear stress, and deflection along the vertical direction of the wall. Also, the soil-cement is designed to resist and redistribute the horizontal stress in the soil-cement between the vertical members, in lieu of lagging, which it replaces.(10)

The soil cement facing analysis consists of determining the maximum bending moment and the maximum shear force per unit width. The minimum targeted compressive, shear strength and modulus for the soil cement is used in the analysis. The internal stress analysis includes shear stress along vertical planes between two H-piles and compressive stress redistribution inside the soil cement facing. The horizontal space between the H-piles is dimensioned to enhance arching and reduce the development of bending stress in the soil-cement. An empirical criterion for spacing reinforcing members to avoid a bending failure of the soil cement has been derived and is shown in Figure 13.(11) Note that L_2 is measured from the edge of the H-beam flange.

For excavation support and groundwater control, a required coefficient of permeability between 10^{-5} and 10^{-6} cm/sec is considered satisfactory. As an example, a coefficient of permeability of $1x10^{-6}$ cm/sec or less is usually required for permanent seepage control in dam, dike, or dry dock projects.

In China, gravity-type DMM retaining walls have been used for excavation support, usually having the area type geometry illustrated in Figure 11 and a total thickness of 0.6H to 0.8H.(24) Constructing multiple parallel rows of columns so that they function as buttresses can provide stability to the column wall using less materials. Then, the un-stabilized soil between the buttresses will contribute to the stability of the wall, with respect to overturning.

In Scandinavia, lime or LCC have been used to stabilize excavated trenches when the depth is less than 4 m (13 ft.).(2) The columns are overlapped to form a continuous wall. The excavation can generally be made about 1 month after the installation of the columns.

The lateral earth pressure acting on the columns is resisted by the weight of the wall in the same way it is in a gravity retaining wall. Bracing should be sufficient to resist potential water pressure behind the wall. As in China, multiple parallel rows have been used.

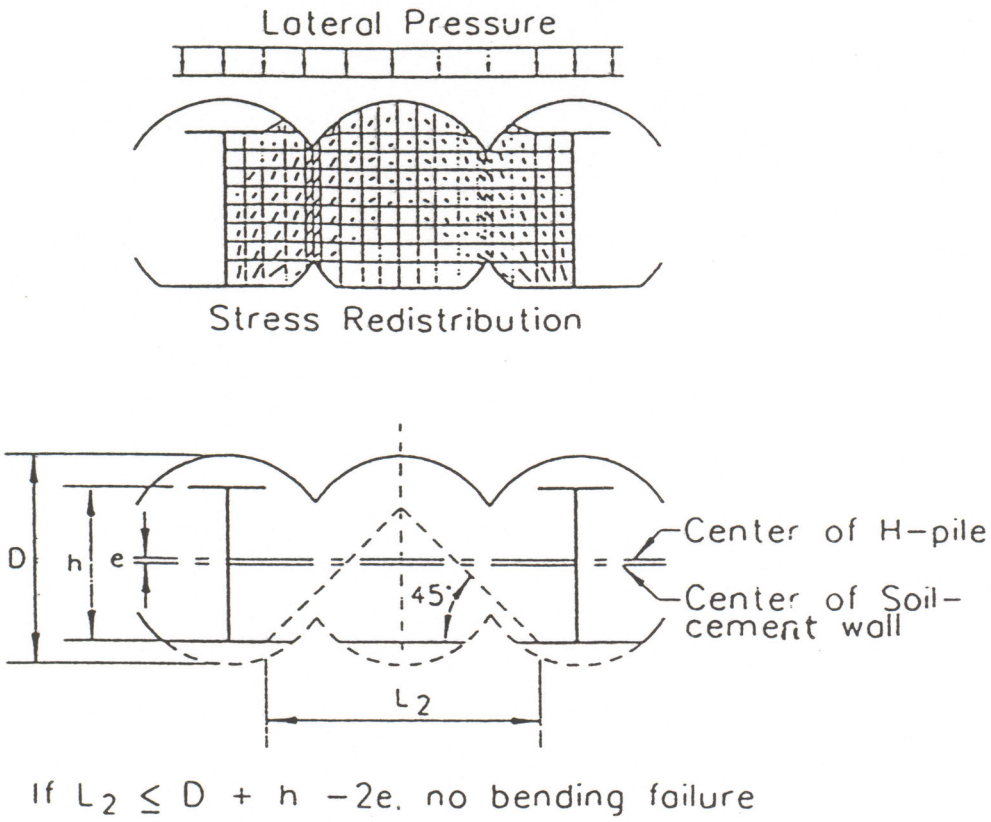

Figure 13. Stress Analysis of Soil-Cement Wall.[11]

b. Stabilization and/or Settlement Reduction for Embankments

The design concept used in connection with applications for settlement reduction and/or stability enhancement is identical to design concepts outlined for stone columns in the companion technical summary. Unit cell concepts are applicable and computations to determine potential settlement reduction and/or average shear strength increase are based on the chosen Stress Ratio, n, the determination of unit pressures on the soil mixed column and adjoining soil with an "optimum" Area Replacement ratio, a_s, which is the ratio of the area of the column to the area of the unit cell. The optimum Area Replacement ratio is obtained by selecting a combination of stress ratio, column diameter, and spacing such that the unit pressure on the adjoining soil has been sufficiently reduced and/or the average shear strength sufficiently increased to meet the project performance requirements.

For settlement reduction, a triangular or square pattern of columns is used, with area replacement ratios on the order of 0.20 to 0.30. Determining the strength to support the embankment load, maximize the benefit of arching between the columns, and provide the required global shear strength to ensure stability develops the optimum design spacing.

Higher area replacement ratios are consistent with more stringent settlement performance requirements. For stability stabilization under embankments, the columns are overlapped and constructed perpendicular to the centerline under the critical areas in a grid-like geometry, typically with a higher area replacement ratio. Successful initial spacing criteria typically uses a clear spacing between elements of 1.5 times the column diameter and a center-to-center spacing no more than 2.5 times the column diameter.

The Area Replacement Ratio a_s, is defined as

$$a_s = \frac{A_s}{A} \tag{1}$$

where A_s is the area of the soil mixed column, and A is the total area within the unit cell.

The stress ratio, n, is a measure of the relative stiffness of the soil mixed column to the in-situ soil; q is the average applied vertical stress; σ_s is the stress on the soil mixed column; and σ_c is the stress on the adjacent soil. For computations, the following relationships are useful:

$$\sigma_s = \frac{nq}{\left[1+(n-1)a_s\right]} \quad \text{(on column)} \tag{2}$$

$$\sigma_c = \frac{q}{\left[1+(n-1)a_s\right]} \quad \text{(on soil)} \tag{3}$$

The total undrained shear resistance τ of the stabilized soil is assumed to correspond to the sum of the shear strengths of the column and the soil between the columns and can be evaluated from

$$\tau = \tau_f a_s + C_u\left(1-a_s\right) \tag{4}$$

Where τ_f is the undrained shear strength of the column, C_u is the undrained shear strength of the soil between the columns, and a_s is the area ratio of the columns. Typical area ratios are on the order of 0.20 to 0.40 and are varied until the targeted minimum total undrained shear resistance of the stabilized soil is calculated. In European practice under the embankment slope and beyond, the columns are constructed in perpendicular panels (rows) and in grid geometries to develop resistance to slope failure. Typically, a Factor of Safety of 1.2 to 1.5 is required to ensure short- and long-term stability.

Scandinavian practice (dry mix) limits the undrained shear strength of the columns used in design to 150 kPa (3100 psf), although higher strengths have been used successfully. Japanese practice (wet mix) limits the shear strength of the columns used in design to 250 kPa (5200 psf). Where significant lateral deformations are anticipated, the undrained shear strength of the column could be significantly reduced to as little as 20 percent of the maximum undrained strength.[15]

The literature contains minimal data on measured stress ratios and their dependence. Typically, they have been estimated as the ratio of the Modulus of the treated to the untreated soil. The stress ratio for a highway project in southeast China, using dry mix cement columns methods, was found to have area ratios between 5 and 5.5.[7] The fill height on this project was on the order of 5 m (16 ft.), and the soft foundation soils were characterized by water contents between 60 – 80 percent and undrained shear strengths from 15 – 30 kPa (300 – 600 psf). The 28-day unconfined compressive strength of the soil mix columns was on the order of 2000 kPa (300 psi), and they were placed at an area replacement ratio of 0.21.

Considerable use of LCC has been made in Scandinavia for settlement reduction and/or foundation shear strength increase in connection with embankment construction over soft soils. Scandinavian practice uses a stress ratio of 4 to 5, which is a ratio based on the ratio of the Modulus of the improved soil to the in-situ soil.

Limited measurements at the I-15 Salt Lake City, Utah, project indicate stress ratios between 8 and 9 for areas with a replacement ratio of 0.4. At this site, the in-situ clays are characterized by moisture contents of 30 – 45 percent and an undrained shear strength of 45 kPa (940 psf). The LCC columns were designed for unconfined compressive strengths between 600 – 800 kPa (80 – 120 psi).

At a recent Virginia DOT test site, stress ratios of 2.5 were measured with a low area replacement ratio of 0.07.

It is anticipated that area replacement ratios of 0.20 to 0.33, and stress ratios of between 4 and 6, would be used typically for this application in either block- or column-type patterns. The limited data to date suggests that stress ratios are dependent on area replacement ratios confirmed by numerical analyses developed by Ogisako[23]. Low area replacement ratios (less than 0.10 – 0.15) are consistent with stress ratios on the order of 2.5 – 3.0.

The reduction in settlement is attributed to the concept that columns that are stiffer than the adjoining soil will carry more load. The load distribution between the treated soil and adjoining soil then depends on the relative stiffness of the columns, with respect to the

surrounding untreated soil and column spacing. This stress ratio (analogous to stone column design) is clearly a critical parameter for determination and/or assumption.[2] At column spacing in excess of four diameters, the stress ratio will approach unity. Once the load distribution has been determined or assumed, an initial settlement estimate can be computed using conventional soil mechanics theory.

The reduction of primary consolidation time is based on the concept that the columns are more pervious than the adjoining soil and, therefore, act as drains. With lime columns, the permeability of the columns is somewhat greater than the adjoining soil, but with lime-cement or cement columns it is often approximately equal.[16]

Based on the above, it has been suggested that for LCC/CC, the effects of radial drainage are possibly "due to possible drainage through a weakened zone left by the installation rod at the center of the columns, together with drainage on the outside of the columns and in fissures and cracks created by in-homogeneities/local over-dosage of stabilization agents in the columns."[16] More recently it has been suggested that the reduction of time for primary consolidation can be attributed to the increase in soil stiffness provided by the columns. Neither theory has been convincingly demonstrated.

Where a significant difference of permeability between the column and the adjoining soil can be demonstrated, time rate computations can be conventionally made using classical radial drainage theory, as applied to wick drains. This difference in permeability should be at least one order of magnitude.

c. Stabilization for Structural Support[6]

For large-scale structural support, a continuous treated soil mass of the wall- or area-type or closely spaced grid-type configuration, as shown in Figure 11, are typical. Current Japanese design considers the mixed soil product not to be a part of the ground. Rather, it is considered to be a rigid structural member buried in the ground in order to transfer the external loads to a reliable stratum. The sequential design steps for mass stabilization are presented graphically in Figure 14. The four main stages of the design are

- *Stability analysis of the superstructure.* In this stage, an improved ground, which is not yet determined, is assumed to have sufficient strength to support a superstructure. Only the sliding and overturning of the superstructure are calculated.

- *Examination of the external stability of the buried rigid structure*; i.e., the mixed soil product, with respect to sliding, overturning and bearing capacity is considered as a

gravity block. The design loads considered are active and passive earth pressures and other external forces exerted onto the boundary of the treated soil structure and mass forces due to the gravity and seismic forces. The allowable bearing capacity of the improved composite soil foundation can be estimated as follows [24]:

$$q_{ca} = a_s Q_a / A_c + \beta[1 - a_s] q_a$$

where:

q_{ca} = allowable design bearing capacity of the composite foundation
q_a = allowable bearing capacity of the surrounding soil
a_s = area replacement ratio
β = bearing capacity factor, 0.5 – 1.0 for friction columns, and 0.1 – 0.4 for end bearing columns
Q_a = allowable design load capacity of a single column; and
A_c = cross-section area of columns.

The ultimate load capacity, Q_u, of a single column can be estimated as follows:

$$Q_u = f_s U_p L + \lambda q_t A_c$$

where:

f_s = average skin friction between the column and surrounding soil
U_p = perimeter of the column
λ = bearing capacity reduction factor for end bearing ranging from 0.4 – 0.6
q_t = end bearing ultimate capacity
L = column length

The average skin friction suggested by the Chinese Technical code based on soil type is as follows[24]:

1. For organic contents > 5% and void ratio $e_o > 1.5$, use 5 – 8 kPa
2. For organic content > 5% and $1.0 < e_o < 1.5$, use 8 – 12 kPa
3. For organic content < 5% and $e_o < 1.0$, use 12 – 15 kPa

The Japan Building Center (JPC) guidelines suggest the following alternate skin and end bearing values[25]:

f_s = c (kN/m^2) for clayey soils, where c is the cohesion

$f_s = \frac{10N}{3}\ (kN/m^2)$ for sandy soils, where N is the STP blow count

λq_t = 6c for clayey soils

λq_t = 75 N for sandy soils, where N is measured at the tip of the column

Typically a FS = 3 is used to reduce the ultimate capacity to allowable.

For design to resist earthquake loads, refer to reference 25.

- *Examination of the internal stability of the buried rigid structure* with respect to internal stress analysis and extrusion (lateral squeeze) failure. Induced stresses are calculated by elastic theory. The size of the buried structure is determined so that the induced stresses are lower than the specified strength of the treated soil. The following design strength are considered:

  (1) Allowable compressive strength = 0.35 – 0.5 q_u /FS
  where FS = 2 to 3
  q_u = average unconfined compressive strength of the treated soil
  (2) Allowable tensile strength = 0.15 allowable compressive strength
  (3) Allowable shear strength = 0.5 allowable compressive strength

- *Displacement analysis of the buried rigid structure.* After the optimum cross section of the improved treated soil mass is determined by trial following the above procedure, the displacement of the improved ground is determined by settlement analyses of the soft layers underlying the treated soil mass.

This procedure is appropriate for the area-type improvement.

The design approach is only conceptually outlined. Analyses approaches, required material properties, and correlation with field performance are under development.

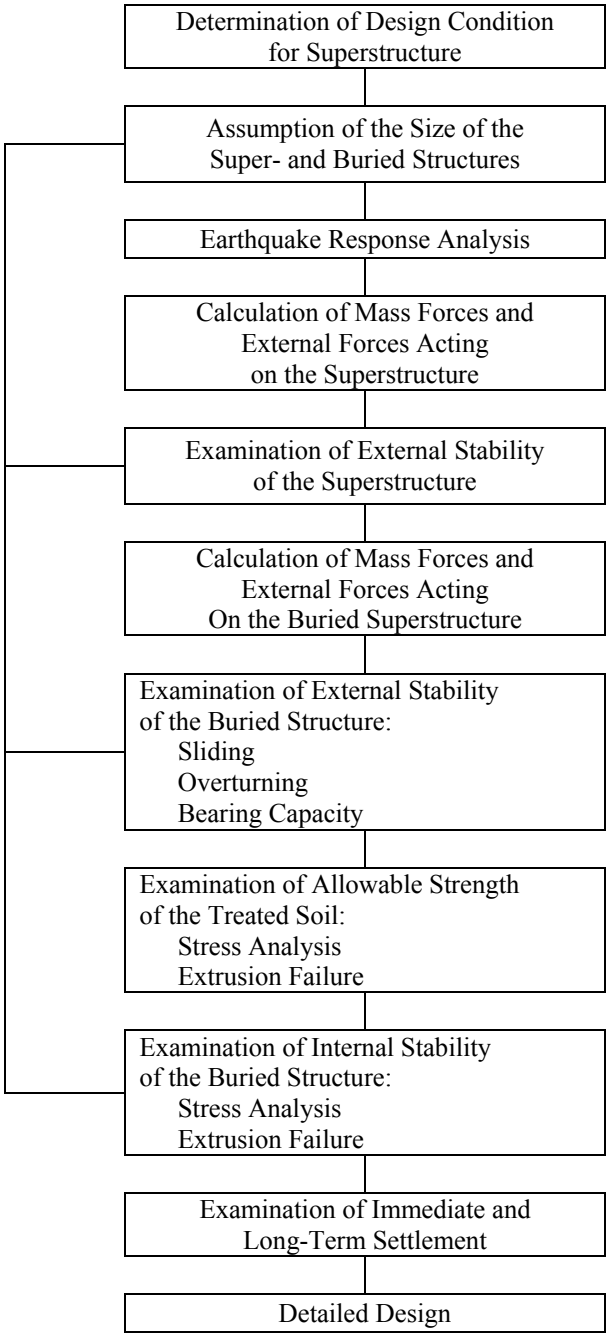

Figure 14. Flow Chart for Design of Continuous Soil Mass.

d. Liquefaction Mitigation

The use of soil mixing for liquefaction mitigation should provide confinement, some reinforcement, and some drainage.

For liquefaction prevention, DMM perimeter cut-off grid walls are installed typically at the toe or under the outer slope of an embankment to confine the loose, cohesionless foundation soils. Mitigation is achieved by constructing in block, wall, or grid patterns, and varying the spacing to provide resistance against internal stresses to preclude horizontal or vertical shear failure and reduce settlement due to spreading. The use of a grid pattern is the most effective, as it confines the entire treated area as a unit for full mobilization of the compressive strength of the composite mixed soil.

Research studies conducted to determine the effectiveness of the grid configuration for reduction of excess pore water build-up during a seismic event indicate that this approach would be effective. The percentage of reduction of pore water pressure has been found to be proportional to the ratio of the space between grid walls to the depth of wall in liquefiable soils. Optimal dimensions with respect to area replacement ratios and depth to width ratios are the subjects of current research.

4.3 SOIL MIX MATERIAL PROPERTIES

a. Wet Mix

The major factors that influence the engineering properties of the composite mixed soil include soil type and chemistry, in-situ water content, amount of reagent used, water-reagent ratio of slurry, degree of mixing, curing environment, construction process and equipment, spoil generated, and age. For excavation support, groundwater control, and soil stabilization applications, the engineering properties that are of major interest are compressive and shear strength, permeability, modulus of elasticity, and freeze-thaw durability.

The strength of the composite soil-reagent can be parametrically evaluated by laboratory testing using unconfined compressive tests, triaxial compression tests, direct shear tests, and tensile tests. The most common is the unconfined compressive test, and its results are used for design and for construction quality control and assurance. It should be noted, however, that the laboratory strengths developed are greater than those obtained in-situ, primarily because of incomplete mixing in the field. It is reported that field strengths, typically unconfined compression, are strongly dependent on laboratory mixing energy and could

range from one-half to one-fifth the strengths determined in the laboratory.[12] Laboratory testing results provide only approximate levels of improvement; however, they are suitable to study the site-specific effects of certain variables, such as water reagent ratio, reagent content, soil type, and moisture content and curing time.

Since the field level of improvement is a function of the enumerated factors and characteristics of the actual mixing equipment used, as well as penetration rate, mixing tool RPM, injection rate and method and withdrawal rate, it is very difficult to predict in-situ strength with accuracy.

The improved strength and modulus ranges indicated in Table 1 have been achieved with the current equipment and methods detailed in Table 3. In general, the engineering properties vary as a function of the following factors:

- *Cement content.* Unconfined compressive strength increases with cement content especially for sand and gravel. The usual range of cement content varies from 100 – 450 kg/m^3 (6 – 28 lb/ft^3) of soil. The relationship between cement dosage and unconfined compressive strength for Japanese soils is shown in Figure 15. Note that Japanese clays typically are at a higher natural water content than clays in the United States.[11]

- *Water cement ratio.* Unconfined compressive strength decreases with increasing water-cement ratio.[12] Minimum water cement ratios are required for workability. Water cement ratios vary from 0.6 to 2, but are more typically between 0.8 – 1.2. There is a tendency among contractors to use lower water-cement ratios when mixing clays and silts to minimize the extra water introduced into the system and higher ratios for sands and gravels. The water/cement-bentonite ratio has the greatest impact on the strength of the mix, rather than the amount of cement. Bentonite will lower the strength and decrease permeability.

- *Curing time.* Strength increases with time. The 28-day laboratory unconfined compressive strength is typically 1.4 to 1.5 times the 7-day strength for clays, and 2 times the 7-day strength for sands.[1] The 56-day unconfined compressive strength may be 1.5 times the 28-day strength.[1] Strength continues to increase with time for perhaps 6 months. Higher in-ground temperatures accelerate strength increase. In-situ strengths may not increase as much as laboratory strengths as a function of curing time.

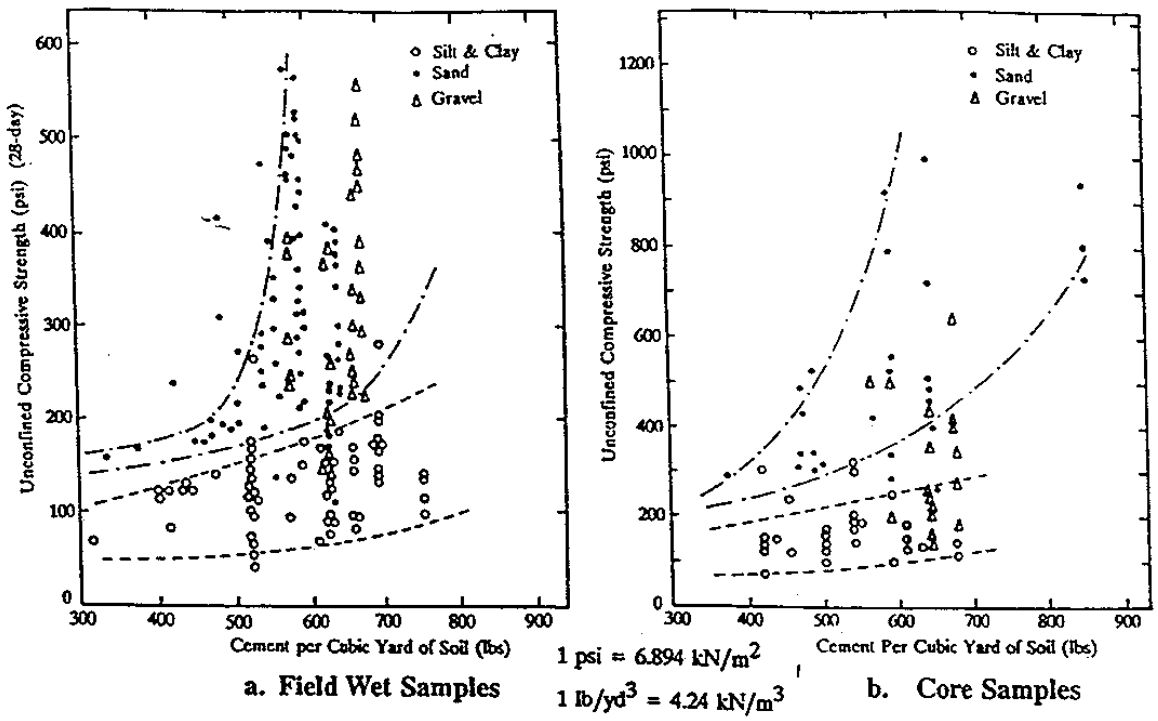

Figure 15. Strength of Soil-Cement.[11]

- *Variability of field strengths.* Significant variability in field strengths, as measured from cores, has been reported. The coefficient of variation from the mean has been reported in Japan to be in the range of 20 – 40 percent, associated with clay soils, and 15 – 30 percent for cohesionless soils.[21] Data from one project in Boston suggest a considerably higher coefficient of variation.[13] This is largely due to the heterogeneous nature of the mixed soil, and the particular construction equipment and process used on this project.

In general, projects using the wet process are typically designed for unconfined compressive strengths of treated soils exceeding 1 MPa (145 psi).

Notwithstanding all the known variables, both natural and selectable, it is reasonable to expect 28-day unconfined compressive strength values as determined by cores for wet mix systems to fall within the broad ranges indicated in Table 4.

Table 4. Typical improved compressive strength, wet mix.

In-Situ Soil	Improved Compressive Strength
Organic and very plastic clays, sludges	Up to 1.2 MPa (175 psi)
Soft clays	0.4 – 1.5 MPa (60 – 220 psi)
Medium/hard clays	0.7 – 2.5 MPa (100 – 360 psi)
Silts	1.0 – 3.0 MPa (145 – 435 psi)
Fine-medium sands	1.5 - 5.0 MPa (220 – 725 psi)

Further

- Air entrainment admixtures reduces strength and increases freeze-thaw resistance.
- Permeability in the range of 10^{-7} to 10^{-8} m/s should be routinely achieved.
- Tensile strength of 10 – 15% of the unconfined compressive strength should be anticipated.
- Shear strength of 33 percent of the 28-day unconfined compressive strength should be anticipated for improved strengths over 1 MPa (145 psi), and up to 50% of q_u for lower strength soil mixed materials.(1)
- The modulus E_{50} should be a function of strength and mixing efficiency. The discrepancy between the laboratory Japanese and Boston Central Artery data shown in Table 1 may be due, in part, to different techniques of measuring strain in laboratory testing.(1) Data developed from field samples from United States projects generally indicates E_{50} varying between 100 and 300 times the unconfined compressive strength. The low end should be considered for excavation support and foundation improvement applications.(1)
- The soil mix material behaves as a overconsolidated clay with a "yield stress" representing the onset of cementation breakdown, similar to a preconsolidation pressure. Loading beyond the "yield stress" would result in large strains. The "yield" stress can be estimated to be equal to the unconfined compressive strength.

To ensure that the design strength is achieved in the field, a performance-type specification should require that the equivalent full column design strength, Q_c, be considered as the design strength σ_c, and could be determined as follows:(21)

$$Q_c = \sigma_c = q_{uf} - m\,\sigma$$

Where σ = is the standard deviation of the strength of the field cores.

q_{uf} = is the mean unconfined compressive strength of the field cores

m = is a function of the standard deviation of field cores for each specific deep mix system and quality control method (1.45 cohesive soils, 1.13 cohesionless soils)

Therefore, the allowable stress in design, f_c using a FS = 3 is

$$f_c = Q_c / 3$$

For preliminary estimate purposes or where standard deviation data is not available, the following relationships may be used:

For cohesive soils	$Q_c = 0.50\ q_{uf}$
For cohesionless soils	$Q_c = 0.64\ q_{uf}$

The above is based on the typical coefficient of variation COV of less than 0.45 for cohesive soils, and less than 0.35 for cohesionless soils.[21] Note that for design, the allowable strength is taken as the design strength, Q_c, divided by a FS of 3 maximum.[21]

b. Dry Mix

The rapid increase in shear strength of soft clay when mixed with lime is triggered by chemical and physical reactions. The reactions include cation exchange, ion crowding, flocculation, agglomeration, and pozzolanic cementation. The extent of these reactions and products formed depends upon the quantity of lime added.[14] Since the strength gain is slow during the formation of these pozzolanic reactive products, cement is added to produce a more rapid gain of strength.

The achieved shear strength is a function of construction equipment and procedures, reagent quantity and type, and the engineering characteristics of the in-situ soil. Figure 16 indicates the level of shear strength obtained in various soils with different quantities of reagent.[14]

Table 5 summarizes the range of anticipated engineering properties that can be achieved with lime-cement or cement columns. The upper strength ranges are usually associated with cement columns.

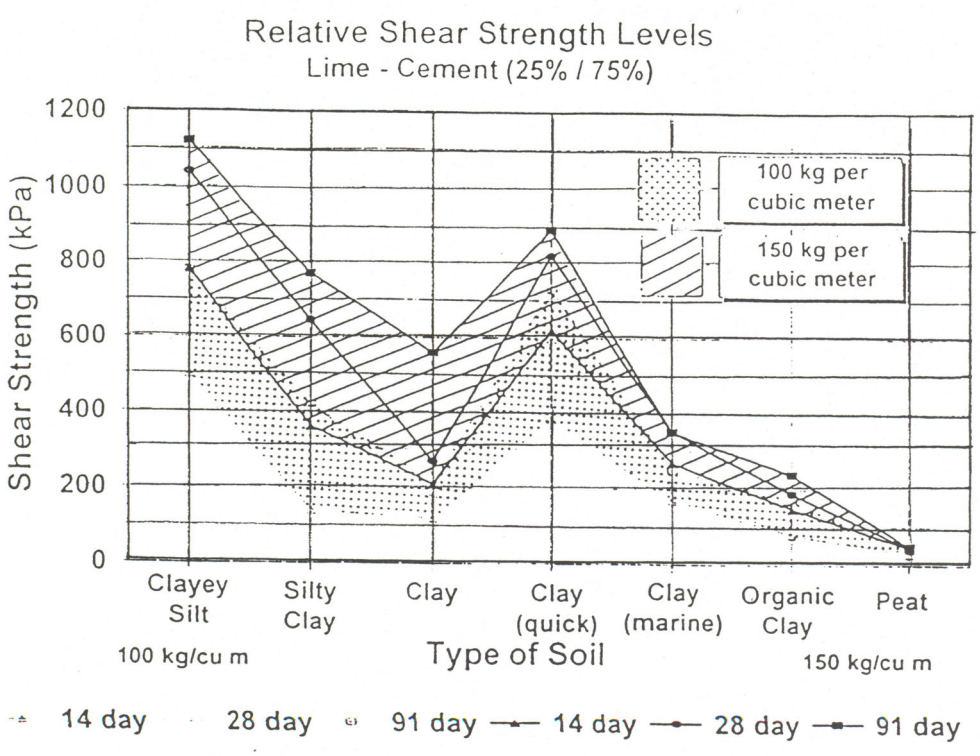

Figure 16. Shear Strength of Different Soils Mixed with Two Quantities of Lime and Cement at Three Curing Times.[14]

Table 5. Typical improved engineering characteristics for lime-cement.

Property	Typical Range
Undrained shear strength, C_u	10 – 50 *x* C_u of soil (150 – 1000 kPa)
Youngs Modulus	50 – 200 *x* C_u (LCC) 50 – 200 *x* q_u of treated soil (cement only)
Strain at failure	< 2 percent
Permeability (lime-cement) Permeability (lime)	about the same as for in-situ soils increases 100 – 1000 times

Typically, 80 – 150 Kg/m^3 of reagent at a ratio of 75 cement/25 lime, or 15% lime and 85% cement is used. The 28-day unconfined compressive strength, typically used for design, will increase from 15 – 30% for most soils in the subsequent two months. The behavior of the stabilized soil is essentially that of an overconsolidated clay, with a relatively high pre-consolidation pressure.

The shear strength of the stabilized soil in the lime-cement columns will not be uniform even after a thorough mixing with the reagent because the installation rod at the center is likely to leave a weakened zone. Other factors affecting the improved soil properties are

- More reagent is required for soils characterized by a high plasticity index.
- Optimum results are obtained when soil moisture content is in excess of 60%.
- Very high soil water content (>120%) reduces improved soil strength.
- The compressibility of the improved soil is reduced with higher preconsolidation pressures.
- In marine clays, the presence of sodium, potassium, and manganese salts is detrimental to strength gain, as is the presence of sulfate reducing bacteria.

As for wet mix methods, shear strength is taken as one-third to one-half the unconfined compressive strength. A maximum shear strength of 400 kPa (8.4 ksf) is typically used, regardless of laboratory results. Currently, significant interest and research is focused on the use of dry mix methods for stabilizing peat and other highly organic soil deposits. The use of significantly larger percentages of cement and additives has resulted in a low strength stabilized soil that may be attractive for certain applications.

Pre-production phase laboratory testing programs are useful in reducing bidding risks and providing a starting point for full-scale field tests and/or pre-production programs. Soil parameters to be tested may include soil activity, water content, plasticity, strength, permeability, compressibility, organic content, and pH. Laboratory variables that affect the engineering properties of the improved soil include

- the initial strength and moisture content of the untreated soil.
- the percentage of reagent.
- the mixing method.
- the curing time.

Dry mixing methods may utilize cement only and are often used to stabilize soils with water contents well in excess of 60%. The reported range of improved engineering characteristics are somewhat lower to those obtained by wet mix methods and are principally a function of cement content.

In summary, it appears that dry mix methods are most attractive when the in-situ soils have high moisture contents, the desired strength of the treated ground is less than 1 MPa (145 psi), and spoil quantities must be minimized.

c. Verification Methods[1]

The properties of the improved ground can be predicted and/or verified by any of the following means:

- laboratory testing of samples before construction
- in-situ testing after construction, as outlined in Figure 17
- laboratory testing of grab samples
- coring and cutting of block samples after construction
- geophysical testing after construction
- other

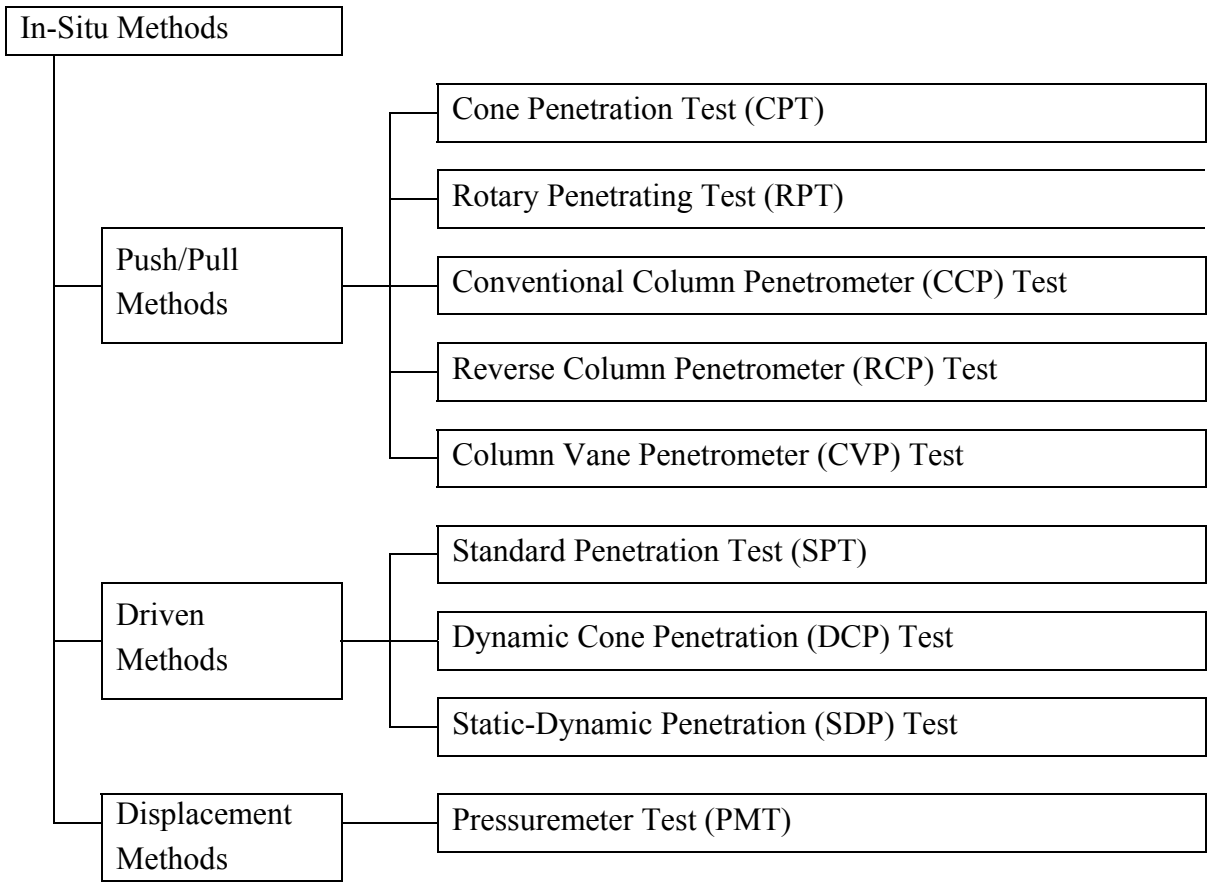

Figure 17. In-Situ Methods for QA Assessment of DMM.[22]

The applicability of these methods is discussed as follows:

Laboratory Testing

Laboratory testing is a valuable tool for confirming basic design assumptions, and for demonstrating the effect and impact of the various reagent materials. These tests are also clearly useful in establishing base line parameters, and for investigating in a controlled fashion the relationships between the various parameters (e.g., unconfined compressive strength and E_{50}; tensile strength; rate of strength gain, etc.). Laboratory testing does not provide a simulation of field mixing and other equipment related parameters, and therefore, is simply an "index" of the actual strength.

In-Situ Methods

Figure 17 summarizes the in-situ methods that have been used. CPT, SPT, and other penetration testing variations appear to be routinely used in Asia. Vane and various column penetrometer tests are common in Scandinavia in connection with relatively low strength dry mix LC methods. Figure 18 illustrates the setup for the pullout probe test on LCC, which is applicable for LCC strengths of less than 200 kPa (29 psi). Pressuremeter testing has been used in the United States to measure modulus and obtain an estimate of shear strength.

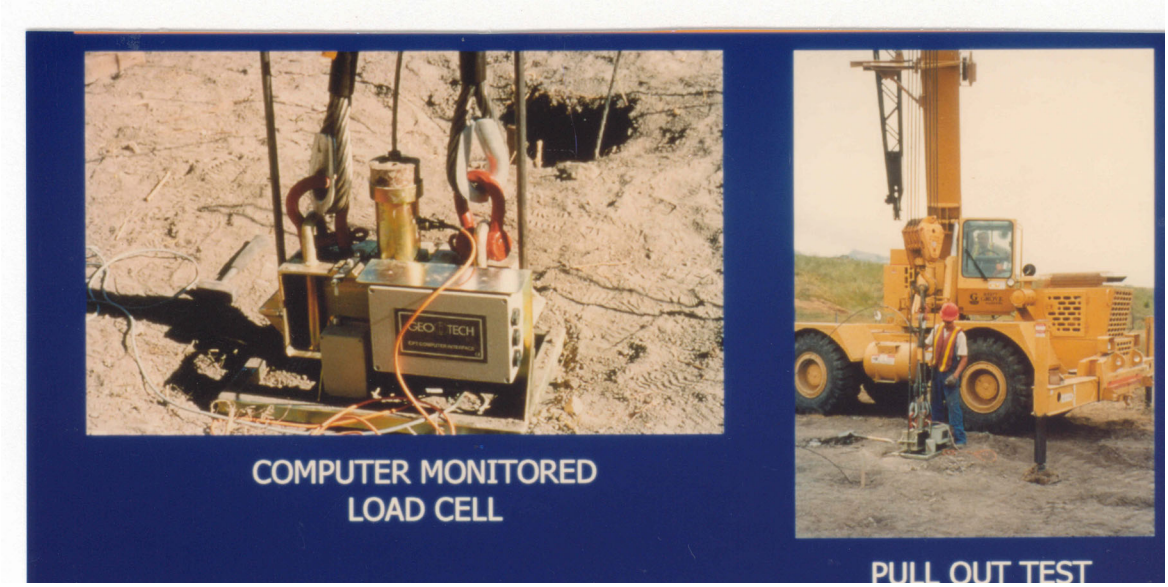

Figure 18. Pullout Probe Testing, LCC.

Coring

Core drilling is often used to obtain test specimens for quality assurance and is becoming the preferred method in the United States. Soil mix columns to be cored are often randomly selected, but additional core drilling and testing should be performed when questionable soil or mix conditions are observed and logged during installation.

Experience in recent projects identified several key elements that promote good and representative core sampling. These include using experienced drillers and logging engineers; taking large diameter cores (greater than 76 mm (3.0 in.) in diameter); using triple tube methods, very coarse diamond bits to minimize sample washout, and appropriate drilling flush (on mud); and ensuring that the inside surface of the sample tube is lubricated. Core samples should be visually examined for continuity and uniformity of the soil-cement mixture, and for strength. Uniformity can be established by testing at sufficient locations with a pocket penetrometer and careful core logging that identifies soft areas and extent. Continuity is defined as the percentage of continuous unbroken, full diameter core. *Continuity of core should be more than 95% in sandy soil and more than 90% in cohesive soil.* Gravels and cobbles can be cause for poor continuity and recovery.

The current Japanese practice for large projects is for a coring frequency of 1 per 3000 m^3 of soil mix product, or at a frequency of between 0.5 – 1.0 percent of installed columns. Most of the improved strengths/values in Table 3 are based on core test results.

Other Methods

Non-destructive testing methods, such as shear wave seismic tomography or velocity, resistivity, low strain sonic and dynamic impact testing, have been used and preliminary correlations developed to core samples.

Non-destructive testing methods are conceptually promising, but insufficient data to judge their accuracy and costs are presently unavailable.[22]

Field Wet Grab Sampling. The concept is to obtain samples from the treated ground before the mix reaches a strength such that a sampler cannot be introduced easily or without causing significant sample disturbance. Typically this requires that the wet mix be at high water/ cement ratios that might not be the most appropriate for obtaining the required strength. Such samples are then used to make cubes or cylinders for laboratory testing. Wet grab sampling presents a number of systematic and logistical problems. For example, the sampling device must be able to reach a prescribed depth, take a representative sample from

that depth, and allow it to be retrieved without contamination. Wet grab sampling requires that emphasis be placed on the efficiency of the sampling tool and the details of the sampling procedure. The potential presence of unmixed native material may prevent the sampler from functioning correctly, and/or from obtaining a wet sample whose composition is representative of the overall mixed volume. Wet samples are screened to scalp material greater than 6 – 12 mm (0.2 – 0.5 in.). The finer material is then used to prepare cylinders for strength testing, which may be 3 inches in diameter and 6 inches in height.

This method should not be used as the project QC or QA verification method for ground improvement projects.

Wet grab sampling could be considered as an adjunct QC method for excavation support projects where visual inspection of the majority of the work is feasible and the work has been procured on a performance contract basis that does not specify a minimum compressive strength.

CHAPTER 5
BIDDING METHODS, CONSTRUCTION SPECIFICATIONS, AND CONSTRUCTION MONITORING

Since both wet and dry DMM technologies are contractor driven and based on specific equipment and methods, performance type specifications appear to be appropriate. Wet projects to date have typically used performance specifications and the single highway dry LCC project was constructed under a method type specification. The LCC I-15 project in Utah is, however, unusual in that it is a design-build project, where the general contractor is responsible for the design, construction, and QA/QC.

5.1 SPECIFICATIONS

This chapter provides project performance specifications derived from a LCC project using dry reagents and a DMM project using wet reagents.

These specifications reflect first time usage of DMM technologies on large projects by two different owners and their engineers. Therefore, the specifications should be considered as a work-in-progress.

a. Wet Deep Mixing Method

Currently, wet technologies have a wide variety of uses. They are used for the construction of seawalls, retaining walls, improving bearing capacity, cut-off walls, treating hazardous wastes in-situ, deep soil densification, and improvement of average shear strength and reduction of settlement.

Due to the large variety of potential DMM applications and results dependent on site-specific conditions, it is very difficult to develop one overall specification that would satisfy the requirements of all projects.

A slightly amended version of DMM or wet specifications used on projects at the Woodrow Wilson Bridge, Route 1/I-95 Interchange in Virginia, is included to form a preliminary basis for development of site-specific specification for ***ground improvement projects.*** Project-specific data or requirements to be developed or potentially omitted are shown in italics. These specifications are quite inclusive and include all aspects of QA/QC programs required.

One essential feature of these specifications is a pre-production or pre-construction Pre-Production (PPC) Program, that establishes construction methods, equipment, mix design, QA/QC programs, and demonstrates the contractor's ability to achieve the required engineering characteristics within the specified tolerances. The elements of this program are outlined in Section 5.2 of this chapter.

Wet Deep Mixing Method (DMM) Guide Specification

SOIL-CEMENT COLUMNS INSTALLED BY WET MIXING METHODS

I. GENERAL

The Work covered by this Section includes furnishing all necessary plant, labor, equipment, and materials and performing all operations of the installation of soil-cement columns by in-situ wet-mixing methods using auger-type equipment, without use of ***externally directed*** high pressure jets. A soil-cement column formed by wet-mixing methods is abbreviated herein as SCCW. The work shall be completed in accordance with the plans, *(fill in DOT's Standard Specifications)* and the Special Provisions.

The SCCWs shall be spaced and arranged as required on the plans and as otherwise directed by the Engineer. The purpose of the SCCW work is to provide embankment stability and reduce settlement of soils under new embankment loads by creating columns of soil-cement made by mixing cement grout with the existing (*fill in soil strata to be mixed*), which in some areas shall form continuous panels of overlapped columns. The Contractor shall install SCCWs to the required compressive strength, to the required depth, and over the required plan areas.

1.1 Definitions

1.1.1. Cement Grout - Homogenous mixture of Portland cement and potable water.

1.1.2. Soil-cement - Mixture of cement grout and in-situ soils.

1.1.3. Soil-cement Return - All materials (spoils) including, but not limited to, liquids, semi-solids, and solids that are discharged above ground surface during soil-cement mixing.

1.1.4. Soil-cement Column (also referred to as Soil-cement Element) – A column of soil-cement formed vertically by injection of cement grout, and the controlled mixing and blending with in-situ soils ***by paddles, augers, and other rigid mixing tools*** that result in a homogeneous mixture of cement grout and the in-situ soils. The soil-cement column may be either one round element or be composed of a series of two, three, or four overlapping and interconnected columns installed by the same insertion and withdrawal operation of the mixing tools.

1.1.5. Obstructions – natural or man-made or man-placed objects or materials occurring at or below ground surface that substantially slows the mixing tools advancement rate, ***as averaged over the full SCCW element depth***, to less than half of the "normal" ***averaged*** rate of penetration ***as determined in the trial SCCW installations***, to the required depths ***in nearby locations*** without having to use maximum downward force ***of the SCCW installation equipment, as defined by the equipment manufacturer's specifications***. Obstructions may include, but are not limited to, rubble, utilities, logs, or tree stumps. Fill and any naturally occurring soil layer will ***not be considered*** as obstructions. ***Obstructions will be classified as either "known" or "unknown" as defined in paragraph 3.5 of this provision.***

1.1.6. Cement Factor - The amount of cement, dry weight in pounds, that remains in the ground after mixing, per cubic yard of in-situ soil-cement, which is also referred to as Residual Cement Factor.

1.2 Contractor Qualifications

1.2.1. The SCCW Contractor shall have a minimum of 5-years' accumulated experience in the work of installing soil-cement columns by in-situ mixing methods similar to that specified herein, over the past 10 years (jet grouting or penetration grouting are not applicable). The SCCW Contractor shall also have completed at least two projects each of which had installations of at least 30,000 cubic yards of soil-cement in similar soil types and depths comparable to those required for the Work specified herein.

1.2.2. The SCCW Contractor's field superintendents shall each have at least three years' accumulated experience in similar work within the past six years including at least two projects in which they installed soil-cement in similar soil types and to depths comparable to those required for the Work specified herein.

1.2.3. The Contractor's personnel responsible for survey layout, lines, and grades, and elevations shall be a Registered Land Surveyor in the *(fill in State*), or a Registered Professional Engineer with at least 5-years' experience in construction layout work.

1.2.4. The Independent Testing Laboratory shall have at least 5-years' experience as a materials testing laboratory, including the performance of testing comparable to that required herein. The person in charge of the testing work for the Independent Testing Firm (I.T.L.) shall be a Professional Civil Engineer, registered in *(fill in State*). The I.T.L.'s supervisor and each field representative who will take samples in the field shall have at least 5-years' experience in taking concrete samples in the field and performing compressive strength tests in accordance with AASHTO requirements, and be licensed as required in the *(fill in State)* to obtain and form concrete test cylinders. The persons who will perform laboratory testing shall have at least 2-years' experience in performing the soil tests required herein.

1.3 Submittals

The Contractor shall submit full information on all aspects of the proposed SCCW work listed below. Allow 23 calendar days from time of receipt for the Engineer to evaluate each submittal. The Contractor shall submit proposed actions and schedule to address any deficiencies within 5 business days after receiving a list of deficiencies. The Contractor shall submit all required data and information at least 30 calendar days prior to the start of SCCW installation. The Contractor shall not commence SCCW installation without the approval of submittals by the Engineer. Approval by the Engineer will not relieve the Contractor of its responsibilities to provide materials and equipment necessary to install SCCWs in accordance with the plans and specifications. If, at any time, the Engineer considers that the Contractor's installation operation does not produce a satisfactory soil-cement column, the

Contractor shall alter its method and/or equipment as necessary to comply with the Plans and Specifications at no additional cost to the Department.

1.3.1. Qualifying Experience

Prior to mobilization of any SCCW equipment to the site, the Contractor shall submit, as listed below, proof of two projects on which the Contractor worked with similar soil types and depths and complexity, and on each of which the Contractor successfully installed at least 30,000 cubic yards of SCCWs, within the last five years.

a. Project Name, location, and completion date.

b. Surface and subsurface conditions, and strength of soil-cement installed (average, ranges, and means used to determine).

c. SCCW installation equipment and technique.

d. Minimum, maximum, and average rates of SCCW installation.

e. Cost and duration of SCCW installation.

f. Average depths and ranges of depths of SCCWs installed and total linear footage of SCCWs installed, and total cubic yardage of SCCWs installed.

g. Current phone numbers and business addresses for the owner/designer, geotechnical consultant, and contract manager.

1.3.2. Schedule

In addition to the requirements of the general provisions and other stipulations provided herein, the Contractor shall submit written notice of intended schedule to the Engineer at least 30 days prior to the installation of any SCCWs, including the trial SCCWs. This schedule should indicate the proposed SCCW installations on a monthly basis through the completion of all required SCCW installations. Also included should be details of daily SCCW work proposed for the two months immediately following each submittal. Schedules are to be

resubmitted in full at the beginning of each month to indicate the SCCW work completed in the preceding month, and all changes to proposed schedule.

1.3.3. Installation Equipment and Procedures

The Contractor shall submit to the Engineer details of the equipment, sequence, and method of installation. The submittal shall, at a minimum, contain the following specific information:

a. Details of the mixing equipment including: size, type, weight, maximum downward force, actual downward force for anticipated set-up and grout mix, type and rated power of source for driving the mixing tools, rotation speeds available for the mixing tools, track sizes and bearing pressure at operating conditions, and configuration of the installation rig.

b. Catalog cut sheets of soil-cement mixing equipment with all pertinent dimensions of equipment and components; cement grout mixing equipment; pipelines; hoses and pumps.

c. Dimensions, configurations, weight, material, and length of the injection and mixing tools, and details of the grout injection ports on the mixing tools.

d. Shop drawings showing how the required overlaps with adjacent elements is to be made, and the layout of each SCCW column/element and identification numbers for each that are to form overlapping buttress. The shop drawings shall meet Specification requirements.

e. Details of proposed cement-grout injection for each column and variations expected within each column, including grout pressure and volume flow rates, auger/mixing tools rotational speed, auger down pressure, auger penetration and withdrawal rates for the soil-cement mix proportions proposed. Include description of expected grout losses from the point of injection to demonstrate that sufficient grout will be retained throughout

the soil column to provide the required/necessary cement factor at all levels.

f. Means that will be used daily to continuously determine the depth of the lowest mixing tool(s), verticality of the mixing tools, cement grout injection quantity at any given depth, and the length of the SCCW installed at each location.

g. Detailed description of all pertinent aspects of proposed installation procedures.

h. Estimated minimum, maximum, and average rates of SCCW installation.

i. Proposed methods for penetrating existing embankment fill or subsurface obstructions.

j. Equipment used for proportioning and mixing cement-grout, as well as all other ancillary equipment.

k. Equipment and apparatus proposed for obtaining wet-grab samples of freshly mixed soil-cement from required depths. Include scale sketches of sampler device that can retrieve soil particles of size indicated in paragraph 1.3.12.b, and calculations to demonstrate workability of means proposed to penetrate sampler to required depths in freshly mixed soil-cement.

l. Equipment and methods to be used for flushing out hoses, pipes, hollow stem auger, tanks, and other grout conveying apparatus that will be used when a rig or plant is shut down, whether at end of shift or in emergency. Indicate discharge areas and holding tanks and ponds.

1.3.4. Details of all working areas from which SCCWs will be installed. At a minimum, this shall include:

a. Proposed plan for constructing level working-surfaces for installation of SCCWs, including drainage provisions and spoil holding ponds or tanks.

b. Engineering analyses to indicate that the proposed locations of equipment, including routes the equipment travel to move about the site or to a site of mixing, do not cause either instability or decrease in strength of either the underlying soils or recently installed soil-cement elements. Design subgrade support to maintain a Factor of Safety of at least 1.5 against both slope instability and bearing capacity failure under the proposed equipment operating conditions and locations, material loads, and fill materials placed as either working-surfaces or stockpile areas during all stages of the Work.

c. Details of all required support equipment storage units, mixing plants, and work platforms, and calculations to demonstrate that loadings caused by any proposed site work or fill configuration on the ground provide adequate factor of safety against instability.

d. Calculations that establish that the working mat fill will not hinder or hamper either the installations of the soil-cement columns, or the penetration of the mixing tools into the ground or their operation, and will not become entangled on the mixing tools or apparatus.

e. Details of all work area organization and sequencing, which shall be scheduled and arranged to prohibit operation of any heavy equipment (such as cranes, trucks, bulldozers, backhoes, mixing machines, and other such machines) for 24 hours or placement of timber working mats on top of any overlapping soil-cement columns that have not reached 7 days of age. Data and analyses shall also be submitted to demonstrate that the loads at any time will not be more than 40% of the soil-cement strength at the time of loading. Show all hose and pipe locations relative to the proposed equipment operations and mixing activities.

1.3.5. Certification of Materials Properties and Handling.
Provide manufacturer's Certification that cementitious delivered materials meet specification requirements, prior to delivery on-site. Provide supplier's Certification that delivered material has been properly stored away from sunlight, moisture, and dirt. Provide all required Source of Materials forms.

1.3.6. Details of all run-off and spoil containment structures shall include: drawings and descriptions of methods to be used to prevent the migration of either grout or soil-cement return spoils, disturbed in-situ soils, or other soil material beyond the immediate limits of the soil-cement mixing operation. Also provide description of processes and procedures to be used to collect and retain the soil-cement return and other spoil materials in such manner to allow the spoils to solidify for the necessary time to become a hardened material resembling a hard, dry cohesive material. The resulting hardened spoils may later be used for general embankment fill, if approved by the Engineer provided the hardened spoils contain no clay particles greater layer than one inch in dimension, and the hardened spoils are hard and brittle and friable, and do not become slippery, sticky, or plastic when wetted. Spoils, which are very hard, and have no indication of becoming slippery, sticky, or plastic when wetted, shall be used as a portion of the "bridging fill" that is to be placed directly over the completed soil-cement columns. In no location shall more than a one-foot aggregated thickness of hardened spoils be placed. Excess spoils meeting the criteria but not used as "bridging fill" may be used as embankment fill. However, wet spoils and spoils that become slippery or sticky when wetted shall not be used, and shall be disposed of off-site, when so directed by the Engineer, at no additional cost to the Department.

1.3.7. Cement and cement grout mix design. This shall include:

a. Cement manufacturer's certificate of compliance.

b. Cement type and water-cement ratio, by weight. Limit water-cement ratio to 1:1 maximum. *(The limit on w/c ratios may be omitted.)* Include details to fully describe and illustrate the methods for grout proportioning to achieve the design mix.

1.3.8. Plans for and results of laboratory 'bench-scale' testing that provide insight as to laboratory mixing characteristics of (*Describe on site soils to be mixed)* that are to be mixed shall be submitted and shall include the following:

a. Laboratory 'bench-scale' testing data to provide insight into the relationship of cement factor and grout water/cement ratio on the 28-day compressive strength of soil-cement specimens, when the soils of the site are mixed in the laboratory. The 'bench-scale' testing will only be used as a general indicator of soil-cement strength that may be obtained in-situ because of substantial differences inherent between laboratory and in-situ mixing conditions. The Contractor shall take appropriate account of these differences in the proposal for trial SCCW installation soil-cement mix design.

b. Prepare test specimens using the full range of soils to be mixed to demonstrate that required 28-day compressive strength will be achieved. Obtain soil samples by means of deep test pits excavated, or other means approved by the Engineer within 15 days of NTP. *(Provide description of potential locations and average index properties of soils to be used for bench testing, especially if organic.)*

c. Upon receipt of NTP, submit within 7 calendar days the details of proposed field sampling and bench-scale testing program for the Engineer to review. Within 10 working days of submittal, promptly commence field-sampling program and undertake and complete laboratory 'bench-scale' mixing and testing program to determine required cement content. *(Provide any further instructions here on the type of soils to be obtained for bench testing.)*

d. Place all soil samples retrieved in heavy plastic bags as soon as dug from ground and seal bags tightly while expelling excess air to thus prevent any loss of moisture. Do not allow field samples of the clay to lose moisture between the time of removal from ground and laboratory mixing. All soil and mixed samples shall be kept out of sunlight at 70 degrees F and

under fully humid conditions throughout storage and curing that prevents loss of sample moisture via evaporation. Test four representative samples taken from different locations throughout each soil bulk specimen retrieved to determine range of moisture content, Atterberg Limits, and organic content by means of respective ASTM tests listed in paragraph 3.2.3. For the laboratory 'bench-scale' testing, the Contractor shall prepare the soil, mix the cement and water to make grout, mix grout and soil together, and prepare specimen cylinders according to procedures submitted to the Department and approved. Test three cylinders of soil-cement at 3, 7, 14, and 28 days following mixing by unconfined compression testing method per AASHTO specification T-208-96, but with strain-rate not faster than 0.5% per minute, but not slower than 0.25%/minute, and with test equipment set up to record in both tabular and graphical form the axial stress and strain constant increments of axial strain no larger than every 0.05% axial strain. Complete within 58 days of NTP all testing for at least three different soil-cement mix designs that could be implemented on each type soil retrieved. *(The lab mixing energy level should be similar to energy levels used by the Contractor's field equipment.)*

1.3.9. Proposed soil-cement mix design shall include cement factor and grout water/cement ratio, and will be reviewed in light of the testing results from the laboratory 'bench-scale' testing. The acceptance of the proposed grout mix/soil/cement mix design shall be contingent on the field trial SCCW installations meeting the acceptance criteria of paragraph 3.7. In the initial trial SCCW installations at each test site, the Contractor shall apply the proposed soil-cement mix design to at least half of the columns (see paragraph 3.2.). However, in the other trial columns at each location, the Contractor may attempt to demonstrate that somewhat different cement factor and/or grout water/cement ratio is workable in achieving required soil-cement strength under actual in-situ conditions.

1.3.10. Submit shop drawings to show plan locations of all proposed soil-cement columns to a plan of scale of 1 inch = 10 feet, and include sequence and time schedule of all soil-cement stabilization operations.

All sequence and overlap details of the soil-cement element installations for stabilizing panels shall follow the sequence shown on the Drawings shall be indicated. The layout of columns shall be based on the required plan area coverage and configurations required on the Drawings. The center location of each proposed soil-cement column shall be uniquely numbered and shown on the plan and dimensioned to the established site-specific coordinate system, noting also any other survey baselines, which the Contractor intends to use for layout and location. For every column/element installed with multiple shaft mixing apparatus, show also the orientation of the axis of the element and its distance from adjacent elements, both those that are interconnected, and others that do not overlap. Show all interconnections and sequences to install overlapping element, to provide the coverage required on the Drawings. The Contractor is advised that in certain locations, the Department may direct use of certain buttress stability panel spacing different from that indicated on the Drawings. When so directed, the Contractor shall modify the layouts of soil-cement columns in the vicinity. If the variations directed by the Department result in an increase in the area coverage from that initially proposed by the Contractor and accepted by the Engineer, then the Contractor will be due additional compensation for the added SCCW area coverage at the contract unit price.

The shop drawings depicting the proposed locations shall be updated at the beginning of each month to indicate the as-built locations for the columns and panels.

1.3.11. Details of supplemental methods and procedures to assist in the installation of soil-cement columns that may be used in the event obstructions (such as buried debris, logs, stumps, cobbles, and utilities) are encountered shall be submitted. If any conflicting buried utilities not located on plans are subsequently discovered, they are to be avoided and protected from possible damage, and the Engineer immediately advised and then consulted as to changes to design of soil-cement elements.

1.3.12. A description of the Quality Assurance-Quality Control (QA/QC) 11/6/2003 Program Plan shall be submitted not less than 21 calendar

days prior to start of trial installation that shall include, but not be limited to the following:

a. Detailed outline of the Quality Assurance-Quality Control Program to be undertaken each day during production to confirm that the soil-cement (SCCW) achieves specified location and overlap requirements, meets the installation criteria, and provides the required strength of soil-cement material. Include all required QA-QC activities and forms and examples of the data to be submitted per paragraph 1.4.

b. Details of procedures to obtain, form, preserve, cure, and transport soil-cement samples. Sampling shall consist of core samples of the completed, cured columns. Submit catalog cuts of the soil-cement curing boxes.

c. Measures to be implemented each day during soil-cement stabilization to monitor, modify, record and control water-cement ratios, cement factors, cement-grout injection pressures and quantities, auger rotational speeds, auger down pressures, auger/mixing tool penetration and withdrawal rates, and other related aspects of the soil-cement stabilization process.

d. Example forms and pages of Daily Production Reports proposed for use and meeting the requirements stated herein. Within two days of completing each soil-cement column, submit to the Engineer as-built field measurement data indicating as-built plan locations of each soil-cement column including the column center (per site-specific coordinates) and the column diameter to the nearest inch; the element longitudinal axis orientation, if made using multiple shaft angles; elevation to the nearest half-foot of the top and bottom of the column, and comparison of the estimated depth of the columns indicated in the Design Summary Table of the Contract Documents to that encountered for each column. The submittal shall demonstrate that the installed columns achieve the required plan area coverage, overlaps, and locations, and that the elements were installed within the verticality tolerances specified.

1.3.13. Independent Testing Laboratory project experience and resumes of the responsible personnel. Submit names and qualifications and lists of experience for all personnel who will supervise or perform field or laboratory work and other documentation that demonstrates the qualifications of each supervisor, field representative, and laboratory technician.

1.4 Quality Assurance-Quality Control Program

1.4.1. The Contractor shall implement a Quality Assurance-Quality Control Program to confirm the SCCW work satisfies the specified performance requirements and acceptance criteria. The Quality Assurance-Quality Control (QA-QC) Program shall be implemented as part of the Work, at no additional cost to the Department.

1.4.2. The Contractor shall undertake general Quality Control for the installation of every column, which shall include real time (computer) monitoring and graphical recording of the following soil-cement mixing parameters continuously throughout the depth of each column:

a. Drilling and mixing depth.

b. Mixing tool penetration and withdrawal rates.

c. Mixing tool rotation speeds.

d. Grout injection rate of each auger.

e. The verticality of each mixing tool shaft in two orthogonal planes.

f. Amperage resistance to penetration.

1.4.3. The Contractor shall obtain, form, preserve, cure, and transport the core samples, including all sample labeling and documentation in the format accepted by the Engineer.

1.4.4. QC Daily Reports shall be submitted for each work shift, within two hours of the end of each work shift. Each QC Daily Report shall document progress of the SCCW construction, present the results of QC parameter monitoring, and present the results of the strength testing of wet-grab samples and continuity of core samples.

1.4.5. The QC Daily Report shall, at a minimum, include the following results of the QC parameters monitored for each SCCW. Each report shall also summarize whether the measured parameters and installations achieved satisfied the Contract Documents: Include in each report the following:

a. Identification of Area of Work, by way of geographic orientation and stations/offsets.

b. Rig number, names of Superintendent and Foreman for each crew.

c. Date and time (start and finish) of each SCCW.

d. SCCW column number, reference drawing number, and coordinates.

e. SCCW top elevation, bottom depth, and total length.

f. Note whether vertical and horizontal alignment satisfy specifications, and listing of any deviations.

g. Final current draw for the drilling equipment at the bottom 2 feet of penetration or final hydraulic pressure, if hydraulic motors are used to turn the mixing tools.

h. Grout injection pressure and volume, estimate of spoil volume being returned per minute; target and actual cement factors and grout specific gravity measurements relative to anticipated quantities.

i. Date, time, plan location, and elevation and other details of all soil-cement wet grab samples and any other samples taken

during the work shift, and the names of persons responsible for obtaining the soil-cement samples. Include test methods and results.

j. Description of obstructions, interruptions, or other difficulties experienced during installation and how they were resolved, and notes or observations relative to cement grout escapes, ground settlement and/or heave, collapses of the soil-cement column, and any unusual behavior of any equipment during the soil-cement stabilization process.

k. Graphical results derived from real time (computer) monitoring for each SCCW at each foot of penetration and withdrawal, and for every minute of mixing tool rotation without vertical motion:

1. Shaft rotation speed and revolution number at each foot of penetration.

2. Penetration and withdrawal rates in feet per minute vs. depth.

3. Quantity of grout injection of each column at every one-foot vertical interval for insertion, mixing, and withdrawal.

1.4.6. Contractor's Daily Quality Control Testing for Soil-Cement Strength

a. The Contractor shall check the specific gravity of the grout at least twice per shift per rig at times and location and depth directed by the Engineer in accordance with ASTM D4380. Report results, including identity of column being installed, depth sampled, and time of sampling, on the shift Daily Report.

b. Soil-Cement Wet Grab Samples: The Contractor may sample using wet grab methods for his own purposes. The Department will not accept results from wet sampling for quality control purposes.

1.4.7. Quality Assurance by Core Samples

a. The Contractor shall obtain continuous core samples from column top to bottom for one column of every 2000 cubic yards of SCCW production, but not less than one column per rig for any week during which the rig is in operation. During the first 5 weeks of production operation of each rig, the number of core holes to be drilled shall be doubled (i.e., 2 per rig per week, or 1 for every 1000 cubic yards of soil-cement installed by each rig, whichever results in a greater number of core holes). The Engineer will designate the columns from which the core samples are to be obtained after reviewing submitted production records. The core samples shall be used by the Engineer to determine if the soil-cement installed meets the acceptance criteria listed in paragraph 3.7. The successful recovery of core samples is also a basis for acceptability.

b. At each core location, continuous core drilling shall commence from the top of the improved soil-cement column, and shall extend the entire depth of the column. Coring shall generally comply with the requirements of ASTM D2113, except that the core barrel shall produce core samples of either PQ or 4-inch diameter, in runs of 5 feet. The coring shall be done only with triple-tube core barrel with split inner barrel, using wire-line equipment. Core bit shall have fine diamonds, be stepped, and have face flush or outward water discharge. The Contractor shall fill each core hole with neat cement grout (w/c = 1:1) at the completion of coring in that hole, using tremie grout methods.

c. Core sample storage and transportation shall comply with ASTM D5079, including the "Special Care" requirements of Section 7.5.2, except that as a moisture loss prevention measure, the Contractor shall promptly (within 30 minutes of removal of core from the core barrel) completely wrap each soil-cement core specimen in two layers of 4-mil-thick clear plastic, and tape the open sides and ends shut to prevent moisture loss. If the Contractor must make further examination of a core specimen, then such examination shall be done in the

controlled environment room with 100% controlled humidity conditions with the specimen exposed for as brief a time as possible, but not for more than 45 minutes.

d. Contractor Documentation of core shall include a detailed field log with sketch and description of soil-cement and soil retrieved from each 5-foot long core run, that illustrates each crack and variation of the composition of the core sample, to a scale of 1-inch to 6 inches of core length and full-scale across the diameter. Provide the completed field logs to the Engineer and make the entire core sample available for inspection immediately at the end of the coring of each core hole. The Contractor shall preserve and retain the core throughout the time of the coring at the column. At the completion of each core hole, the Contractor shall transport the core specimens of the completed core boring to the Engineer's field office where the Engineer will select portions to be tested in the Engineer's laboratory for unconfined compression strength. In total, the Engineer will select 5 to as many as 10 specimens from the entire length of the core specimen, generally all from depths of the soil-cement column length in the alluvial clay. The Engineer will select specimens from as many as three depths of any retrieved 5-ft. core run. The Contractor shall wrap each specimen the Engineer selects in two layers of 4-mil plastic and tape to prevent loss of moisture, and label each specimen with soil-cement column number and depth. The Contractor shall transport the specimens in a padded wooden box, which provides padding between each specimen, to the Engineer's testing facility laboratory within 12 hours of the Engineer's review. The Engineer will complete unconfined compression tests of the core specimens using AASHTO Test T-208-96 and will retain the remaining parts of the selected core specimens. The Contractor shall retain all remaining core not selected for testing in a heated and humidified storage room where minimum 95% humidity and 70 degree F shall be continuously provided throughout the remainder of the project time.

e. If core recovery is less than either 85% of the total measured length of core hole drilled, or less than 80% each for any two

of the 5-ft. long core runs in the depth of the clay soils stratum, then the Contractor shall drill another core hole in a nearby SCCW element as directed by the Engineer, at no additional cost to the Department. If the core recovered from the second core hole does not provide core specimen over the elevations of missing core from the first core hole, then the Contractor shall drill yet another core hole as directed by the Engineer, at no additional cost to the Department.

f. If three successive core holes each obtain less than 85% recovery, or any two fail to obtain core in the same elevation horizon within the clay soils stratum, then the soil-cement mixing operations shall be considered non-compliant, and suspended immediately. If any core fails the acceptance criteria, then the column shall be deemed non-compliant. In the event that the soil-cement mixing operations are suspended for non-compliance, all of the soil-cement columns installed by that rig following the installation of the last soil-cement column that was core drilled and determined to be compliant shall be considered non-compliant. The Contractor shall not resume soil-cement mixing until it is demonstrated by drilling additional core holes that the soil-cement installed has the consistency, uniformity, and strength required in the Acceptance Criteria in paragraph 3.7. The drilling of additional core holes shall be at no additional cost to the Department. No further soil-cement mixing shall be performed until the Engineer is satisfied with the Contractor's substantiation that all of the soil-cement installed does meet all specified requirements and acceptance criteria. There shall be no payment for soil-cement designated as non-compliant. Any delay resulting from the Contractor's non-compliance will not be the responsibility of the Department and cannot be cause for claim by the Contractor.

1.5 Project Conditions

Prior to bidding, the Contractor shall visit and examine the work site and all conditions thereon and take into consideration all such conditions that may affect this work, in accordance with Section *(Fill in)* of the Standard

Specifications. Subsurface data is available for review at the (*Fill in*). Other project site conditions of pertinence are as follows:

1.5.1. Protection of Existing Structures: Protect structures, underground and overhead utilities, and other construction from damage caused by SCCW installation. The Contractor shall be responsible for all damage or movement to embankments, abutments, pile-column bents of viaduct and any other existing structure, including settlement or heaving of ground or lateral soil movement that could occur due to SCCW stabilization in the immediate vicinity of existing structures, roadways, viaducts, or utilities.

1.5.2. Throughout execution of the Work, the Contractor shall take all precautions and measures necessary to safely move and position SCCW stabilization equipment, support equipment, and personnel around the site. It is anticipated that SCCW mixing will be performed simultaneously with other soil stabilization methods and earthwork activities. The Contractor shall provide equipment, materials, cribbing, timber mats, or other support structures necessary to provide a stable-working surface for SCCW stabilization. If granular fill is used, it shall not raise the site grade to elevations more than 2 feet higher than the construction preparation conditions indicated on the drawings, without specific engineering analyses, which the Contractor shall make to demonstrate that there is a factor of safety of at least 1.5 against any form of instability, and prior approval of the Engineer. The Contractor shall be responsible for evaluating and analyzing the soil and soil-cement stability based on the Contractor's proposed construction sequence and methods.

1.5.3. The Contractor shall cooperate and comply with all local, state, and federal agencies or authorities working or having jurisdiction at the site.

1.5.4. The Contractor shall coordinate all SCCW installations with other utility work required, in accordance with sequence and time schedule requirements contained in the plans and specifications. SCCW installation in the area of the 60-inch jacked pipe shall be completed prior to installation of the pipe extension.

b. Lime-Cement Columns

The I-15 Utah specifications are included in slightly amended form to serve as a guideline for specification preparation. The description section should be site specific and for a performance-type specification, a section on Performance Requirements or Acceptance must be added to detail the geometry of construction and the required engineering characteristics of the lime-cement columns. This should include a specific and clear explanation of the required QA/QC (methods and frequency) of monitoring, as well as a specific definition of acceptance criteria. Also a measurement and payment section must be added.

Lime-Cement Columns Guide Specification

1. Description

The work covered by this section includes furnishing all plant, labor, and equipment, and performing all operations required for installing and testing lime-cement columns to stabilize embankments, walls, or other facilities, as specified herein and shown on the contract drawings.

Lime-cement columns will be installed to stabilize foundation soils along the *(describe project)*. The columns will support selected embankments, retaining walls, and other structures associated with the new roadway. Lime-cement columns are intended to: (a) strengthen natural soils, allowing single-stage construction of embankments and walls, (b) significantly reduce primary consolidation settlements, and (c) reduce or eliminate secondary compression within the depth of treatment. The Contractor shall familiarize himself with project geotechnical conditions and recognize that *(describe key geotechnical features including groundwater location, availability of geotechnical report, boring logs, and other pertinent information)*.

1.1. Contractor Qualifications

The Lime-Cement Column Contractor shall demonstrate a minimum of 5 years' experience with the installation of lime-cement columns, and shall provide at least one operator for the lime-cement column equipment with a minimum of 5 years' experience with the equipment and with lime-cement column construction.

1.2. Submittals

No less than 30 days before production level of lime-cement column installation is to begin, the Contractor shall submit the following for approval by the Engineer.

A. Detailed experience profile to demonstrate that the requirements for 5 years of experience with lime-cement column installation have been met.

B. Experience and/or training records of operators of lime-cement column equipment and of on-site supervision of the Contractor's operations.

C. Results of preconstruction laboratory and field testing with a final written report.

D. Sample log to be used to record column installation. The log shall contain at least the following information:

- Project Name.
- Machine Number.
- Type of mixing tool.
- Date and time of column installation.
- Column number and reference drawing number.
- Column diameter.
- Column length.
- Quantity of lime and quantity of cement injected in kg/m.
- Installation air pressure at tip and top of the lime-cement column.
- Rate of withdrawal of mixing tool in mm/revolution.
- A description of obstructions or interruptions of lime-cement injections during installation.

E. Supplier's certifications of lime and cement quality *(and other additives, if used)*.

F. Description of method(s) and equipment to be used to penetrate existing embankments and obstructions and to install lime-cement columns.

G. Metering equipment calibration test results.

1.3. Delivery, Storage and Handling of Materials

1.3.1. Quicklime and cement shall be stored in closed pressure tanks suitable to be used as pressure vessels, for all pressures required. including those used to load and unload the materials.

1.3.2. Delivery trucks shall be loaded at the manufacturer's plant unless approval is given for an intermediate storage facility. Each truck shall have a certified record of the weight of each load of material. The material shall be transported to the project site and blown into the on-site storage tanks using a pneumatic system. The air evacuated from the storage tanks during the loading process shall be filtered before being discharged to the atmosphere.

1.3.3. A sealed refilling machine shall be used to transport material from the storage tanks to the lime-cement column machine. This machine shall be refilled using a pneumatic system and an air filter, as specified above.

2. Materials

Standard Type I-III Portland cement shall be used or as approved by the Engineer.

Quicklime shall have at least 99 percent passing the #8 sieve (3.18 mm) and at least 90 percent passing a No. 12 sieve (2.12 mm), an active CaO content greater than 80% and a floatability of 70.

If premixed quick lime and cement is to be used, the manufacturer of the mixture must certify that the proportions of lime and cement provided are in accordance with design drawings.

3. Installation Equipment

3.1. Lime-cement columns shall be constructed using computerized self-contained lime-cement column machines. The equipment at a minimum shall be comprised of:

- 2 pressure tanks
- Telescopic leads or fixed leads capable of firmly supporting the injection pipe in a vertical position throughout the installation process.
- Available torque of 1.8 m (5.9 ft.) and rotational speed of up to 160 rev/min.

- Compressor (8 bar)
- Base machine with crawler tracks to provide low ground pressure
- Pressurized operator cabin to avoid dust within the cabin.
- Specially designed mixing tools with the capacity to install columns ranging from 600 – 800 mm (24 – 32 in.) in diameter and to a depth of *(fill in depth)*.
- Computer system for monitoring and recording installation data.

3.2. Refilling machine shall be on wheels and shall contain:

- Two tanks or one divided tank
- Compressor

3.3. Storage tanks shall be:

- Tanks or Silos with adequate storage for continuous production
- Tanks shall be equipped with air filters

(Under a performance-type specification specific, equipment requirements should be omitted.)

4. Installation Procedures

4.1. Preparation

4.1.1. The presence and location of buried pipes, sewers, and other utilities shall be identified and precautions taken to protect them from damage by the installation of lime-cement columns. The Contractor shall be responsible for damage.

4.1.2. Lime-cement column limits and locations shall be established by survey. Individual column locations shall be marked. Sufficient vertical control shall be provided to establish that lime-cement columns reach plan depths.

4.2. Installation

4.2.1. The installation of lime-cement columns shall be in the patterns and with the proportions and weights of lime and cement and at the injection pipe withdrawal rate required to meet the design column strengths, as shown on the design drawings. All lime-cement column panels shall be constructed in such a manner

that adjacent columns provide the minimum overlap shown on the plans for the full depth of adjoining columns.

4.2.2. Installation of each column shall be continuous. Deviations from plan locations at the ground surface shall not exceed 100 mm (4 in.). Deviation from vertical at the ground surface shall not exceed 1%.

4.2.3. The preset data in the on-board computer shall be verified as correct for each column and, if necessary, adjusted. The operator shall monitor and adjust as necessary during column installation the feeding of material, the injection air pressure, and the rates of rotation and rise. All metering equipment shall be calibrated at least every 100,000 m (328,000 ft.) of column installation, and the results supplied to the Engineer.

4.2.4. Lime and cement shall be injected by air pressure occurring as the mixing tool is raised. The tool shall mix the reagent with the soil and water to construct the lime-cement columns.

4.2.5. Injection of lime and cement binder during column installation shall terminate at the design elevation shown on the plans.

4.2.6. Unforeseen conditions that result in changes in the rate of rotation or rise of the injection pipe, the rate of lime-cement binder injection, or other critical factors during column installation shall be noted on the installation log and deviations from the standard corrected as follows:

- Interruption in the process of column installation shall be corrected by redrilling the lime-cement column a minimum of 300 mm (11.8 in.) below the elevation of the interruption and restarting the injection and the mixing of lime and cement, while raising the mixing tool to the preset stop elevation.

- Interruption of the installation process because of obstructions or a dense layer above the planned tip elevation shall be immediately brought to the attention of the Engineer. The mixing tool shall be removed from the excavation. Obstructions shall be penetrated with auger drilling equipment or other approved methods to remove the obstruction(s) or to loosen the obstructions, including any dense sand layers, sufficiently to allow the installation of the lime-cement column, unless otherwise indicated by the Engineer. Where the obstruction cannot be penetrated, the lime-cement column shall be completed

to the maximum depth penetrated. The need for an alternate design or remedial construction shall then be determined by the Engineer.

4.2.7. A log of the installation of each column shall be prepared by the lime-cement subcontractor and submitted to the Engineer within two days. The log shall contain the information listed in Section 1.2.D.

4.2.8. Columns shall penetrate to a predetermined design elevation. If the maximum depth is less than the design depth, the provisions of Section 4.2.6 shall govern.

4.2.9 Natural soils above the water table shall, at the completion of lime-cement column installation, have been treated to produce the full column design strengths within the clay soils up to within 1.0 m (3.3 ft.) of the ground surface. *(Existing embankment soils or sand layers above the water table shall have been treated such that the soil strength up to within one meter of the ground surface is at least as great as it was prior to column installation.)*

Upon completion of the column installation the ground surface shall be scarified and compacted as embankment in accordance with the Standard Specifications.

4.3. Acceptance Criteria

4.3.1. Lime-cement columns shall be considered acceptable when (a) the columns have been installed to a predetermined design elevation or to the satisfaction of the Engineer, (b) the recorded length of the column has been verified as correct by the Engineer, (c) the continuously recorded quantity of lime and cement injected during column installation has been verified as within 10% of the design (preset) value shown on the plans, (d) the location of the top of the columns has been verified as within design tolerances, and (e) *the design shear strength of the columns has been verified by tests of actual columns.*

5. Quality Control

5.1. Test Columns

A minimum of 0.5% of production columns (1:200) will be subjected to a probe test *(or other test method established by the Engineer)* to establish continuity of the columns and to obtain a measure of their strength, unless otherwise directed by the Engineer. Probe testing shall be done on selected columns throughout the

area being stabilized to provide a continuous record of force with depth. The probe shall be 400 – 600 mm (16 – 24 in.) in width and 15 – 20 mm (0.59 – 0.79 in.) thick. Testing shall be done in accordance with the Swedish Geotechnical Society, SGF Report 4:95E, 1997. *(Note that this method is only applicable to low strength columns)*.

5.1.1. Probe tests shall be performed on columns cured for a minimum of 3 days, or as directed by the Engineer. Test columns for probe testing shall have been installed in the same manner as a remainder of the production columns, unless directed otherwise except that these columns will have had the inverted probe installed at least 1 meter below the tip of the column before the mixing tool is raised and the column formed. Test columns that have been probe tested shall not be considered as production columns.

5.1.2. Prior to full-scale production, test lime-cement columns shall be installed at each location to be treated, to verify installation procedures, to confirm design strengths, and to ensure that the installation equipment can install lime-cement columns to design tip elevations, as shown on project plans. Three test columns, with probes, shall be installed at each location and tested 3 days after column installation, or as directed by the Engineer.

If 40% of the design strength has been gained during that period, then the next column will be tested at 5 days after installation. If 40 percent of the design strength has not been achieved, a second probe will be pulled at 3 days. If 40 percent of the design strength is not reached in the second probe test, the third column will be tested at 5 days after installation.

Failure to achieve 60 percent of the design strength at 5 days will require changing cement and lime quantities and re-initiating of the testing program. Where obstructions are found, the obstructions shall be penetrated with methods selected by the Contractor, and the lime-cement column completed to the plan depth. Test column locations shall be identified by the designers and the access coordinated with the Contractor. The use of lime-cement columns and the need for special methods to remove obstructions shall be determined from the results of test-column installations.

Production columns shall be installed in accordance with the means and methods used for installation of the accepted test columns.

(The in-situ shear strength of the columns may be tested by conducting pressuremeter testing. Alternately, complete columns can be recovered by driving a square 11 m (36.1 ft.) long sampler, by vibratory methods, and recovering whole columns of stabilized soil for laboratory testing, or by coring methods.) (A verticality requirement may be added if LCC are used for excavation support.)

6. Measurement and Payment

(Lime-cement columns may be paid on an L.M. basis as total compensation for equipment, material, and labor required for installation.)

2.0 MATERIALS

2.1. Cement Grout

Cement grout shall be a stable homogeneous mixture of cement and water. Cement grout mixture is not to be changed unless conditions encountered so require, in which case the Contractor shall request in writing with substantiating reasons a change, subject to the approval of the Engineer. The Contractor shall not change cement grout composition without approval of the Engineer.

Cement shall conform to AASHTO M85, Type II Portland cement, and as specified in the submitted cement grout mix design, accepted by the Engineer. Type III Portland cement shall not be used. Neither slag cement nor fly ash shall be allowed without the acceptance of the Engineer of submissions made by the Contractor of test data that confirm no deleterious impact to the soil-cement *(including, if applicable, consideration of the variable range of organic content and possible effects of sulfates and formation of ettringite)*, and which indicate in detail the cementitious effect of the proposed product and comparison to the specified Type II Portland cement. Measure, handle, transport, and store bulk cement in accordance with the manufacturer's recommendations. Cement packaged in cloth or paper bags shall be sealed within plastic or rubber vapor barriers.

2.1.1. Store cement to prevent damage by moisture. Material which has become caked due to moisture absorption shall not be used. Bags of cement shall be stacked no more than ten bags high to avoid compaction. Cement containing lumps or foreign matter of a nature that may be deleterious to the grout mixing or delivery or injection operations shall not be used.

2.1.2. All cement shall be homogeneous in composition and properties, and shall be manufactured using the same methods at one plant by one supplier.

2.1.3. Tricalcium aluminate content shall not exceed 7%.

2.2. Water

Water used in drilling, mixing cement grout, and other applications shall be clean and potable, of neutral pH, and free from sewage, oil, acid, alkali, salts, organic materials, and other deleterious contamination. If the Contractor obtains water from sources other than public drinking water supply, then the Contractor shall submit the results of water quality testing done on a daily basis to demonstrate that the water is clean, of neutral pH, and is free from sewage, oil, acid, alkali, salts, organic materials, and any other deleterious contamination.

2.3 Admixtures

Cement grout admixtures shall not be allowed, without prior submittal showing any or all effects on final soil-cement mixtures and long-term character of the in-situ soil-cement, and the subsequent review and acceptance of the Engineer.

2.4. Soil-Cement Mixture

2.4.1. Soil-cement mixture shall be a stable homogeneous mixture of cement grout and in-situ soil. Proportion the amount of cement grout mixed into the soil to be sufficient to overcome all conditions found in the test borings and the test pits, and for other conditions as will be encountered throughout the site during construction. The composition of the soil-cement shall be verified throughout the course of the Work in accordance with the Contractor's Quality Assurance-Quality

Control Program, as implemented by core recovery, core log records of soil-cement composition and uniformity, field probe testing of core samples, and the Engineer's laboratory testing of samples from core borings. The Contractor's testing of wet grab specimen cylinders, if any, will be used only as a guide and advance indicator of probable soil-cement strength.

2.4.2 If the 28-day unconfined compressive strength for any core specimen tested does not meet the acceptance requirements for strength, or if uniformity or core recovery are not acceptable, then all SCCW installed since the previous acceptable tests and examinations shall be considered non-compliant, until the Contractor obtains additional core samples from columns designated by the Engineer, and subsequent tests by the Engineer indicate adequate uniformity and strength. These additional core borings made by the Contractor to retrieve confirmatory core specimens that Contractor shall supply to the Engineer for manual-visual classification and strength testing shall be considered as a supplemental part of the QA program, but shall be made at no additional cost to the Department. If it cannot be verified that soil-cement meets strength and uniformity requirements, then the Department may require installation of additional columns, at no additional cost to the Department.

2.5. Working Mat Fill

(*Describe, if necessary, including location and quality.)*

3.0 EXECUTION

3.1. General Equipment Requirements

3.1.1. The Contractor shall provide equipment as required for installation, sampling, and testing of SCCWs in accordance with the Plans and Specifications.

3.1.2. All equipment shall be maintained so as to ensure safe, continuous, and efficient production during SCCW stabilization and other related operations.

3.1.3 All equipment shall have real-time (computer) controls to permit accurate and continuous monitoring, recording and control of the following: cement and water proportioning, grout mixing, water-cement ratios, mixing tool depths, cement factors, cement-grout injection pressures and quantities, auger rotational speeds, auger down pressure, auger advancement and withdrawal rates, amperage developed, and other operations required to install the SCCWs.

a. Injection volume estimates shall be only made by precision in-line flow meters. Counting or measuring grout pump strokes shall not be acceptable.

b. Injection quantities must be measured in real time by direct measurements of volume and/or mass for each column shaft having injection capabilities, with flow meters and other measuring equipment having precision accuracy not less than 99.5%. Gages and flow meters and other measuring equipment shall be calibrated and certified as precise and accurate before the start of the equipment's work on the project, and then again every 4 months.

3.1.4. SCCW stabilization equipment shall be of sufficient size, capacity, and torque to perform the required SCCW stabilization operations. The equipment shall be capable of advancing through previously installed and cured soil-cement elements as necessary for installing overlapping ***and end junction*** elements. The equipment shall utilize sufficient mixing and injecting equipment to adequately blend cement grout with the in-situ soils to produce a homogeneous distribution of cement grout throughout the mixed in-situ soils that meet the acceptance criteria of paragraph 3.7.

3.1.5. SCCW auger mixing equipment shall form SCCWs of the required diameter and size as submitted by the Contractor and reviewed and approved by the Engineer. Mixing tools shall uniformly inject cement grout through hollow stem or other piping at locations that distribute the grout across the full diameter of the mixing tools and such that the full auger/mixing paddle assembly passes through the column of soil after the grout is introduced, on both the insertion and withdrawal strokes. ***Grout shall only be injected in direction within the diameter***

of the augers or mixing paddles. ***If grout injection jets are used, they shall not spray beyond the auger diameter***. Continuous auger flights longer than 3 feet or with more than one full, uninterrupted revolution of auger are not allowed as part of the mixing tools. Auger flights and mixing paddles on a shaft shall all reach to the full column diameter, and shall have discontinuous lengths and be so oriented as to thoroughly break up the in-situ soils, and disperse and blend soils with injected cement grout to form a homogeneous soil-cement mixture.

3.1.6. SCCW mixing machine shall have inclinometer-type instrumentation to measure and record vertical alignment of mast and/or leads to confirm maintaining verticality of guide holding the mixing tools and motor. Instrumentation shall be capable of measuring deviations from verticality to an equivalent of 1-inch in 100 feet. Verticality shall be monitored and maintained both fore and aft, and left and right of the machine.

3.1.7. The cement grout batching plant shall include all storage silos and sheds, pumps, scales, mixers, valves, gauges, and regulating devices required to continuously measure and mix cement grout in real time.

3.2 Trial SCCW Installation

3.2.1. Prior to the installation of the production SCCWs within any of the required areas for soil-cement mixing shown on the plans, the Contractor shall demonstrate that its equipment, method, and materials produce satisfactory installations in the soils of that particular area, in accordance with the plans and specifications. Each Contractor proposed to undertake soil-cement column mixing shall successfully complete a trial installation for the area of work, as designated by the Engineer. The Contractor shall install trial soil-cement columns in areas designated by the Engineer, which will be in the immediate vicinity of production SCCW locations. The Contractor may propose that some trial SCCWs be at production locations, but not more than half of the trial columns in any one area, and only as reviewed and accepted by the Engineer at the Engineer's own discretion. Those trial columns accepted by the Engineer to be at production locations shall be installed at the end of the installation at one area, and may only be installed using the higher of proposed cement factors and only after the

first columns installed have had no apparent problems and core samples have been obtained as required and have provided acceptable 7-day strength. The Engineer may at any time during the trial installation require all trial SCCWs to be installed at non-production locations.

3.2.2. The Contractor shall install (*Fill in number and indicate location)* SCCWs as trial columns at each of the different test areas. A trial column will be counted as an insertion and withdrawal (or multiple passes) of either a single-axis or multiple-axis mixing machine, with no reduction made in the number for multiple-axis mixing tool set-ups. At least one-half of the trial SCCWs shall be installed by the same procedures and grout mixture as proposed by the Contractor for the production soil-cement columns. The other columns may be used to test alternate procedures and grout mixes, but these shall only be located at non-production SCCW locations. (*The Contractor shall install one-half of each group of columns as overlapped in accordance with requirements of the plans.)*

3.2.3. In each of the areas selected by the Engineer for the SCCW trial installations, the Contractor shall within 21 days of receipt of NTP, have two test borings drilled to retrieve continuous split spoon samples of all soils strata to be mixed. Use Standard Penetration Test methods, ASTM D-1587, to advance a test boring near the center of each area of trial columns and retrieve continuous samples, and prepare detailed log of each test boring for submittal to the Engineer. Test representative portions of each split spoon sample of soil for the following classification tests: Atterberg Limits by ASTM D-4318; Moisture Content by ASTM D-2216, and Organic Content by ASTM D-2974. Perform additional series of classification tests if differences are apparent in the clay soils retrieved in a given split spoon sample. If the area for the trial installations encompasses more than 700 sq. ft., then the Contractor shall make one additional test boring for being over 700 sq. ft. and one more boring for each additional 500 sq. ft. area. The Contractor shall submit the results of the laboratory classification tests to the Engineer at least 10 working days prior to the commencement of the installation of the trial columns at an area.

3.2.4 At each trial SCCW installation area, the Contractor may obtain wet grab samples from no fewer than six of the trial soil-cement columns installed by each of the different procedures and/or grout mix and/or cement factor, and shall test cylinder samples from each, all in accordance with the QA/QC protocols. The Contractor shall also drill core holes in 6 SCCWs installed by each of the different procedures and/or grout mixes and/or cement factor. The Engineer will determine which SCCWs are to be core drilled (which may include columns from which wet grab samples were retrieved) based on observations during installation and data provided by the Contractor. The core drilling shall be done according to procedures in paragraph 1.4.7.b., and specimens from both the wet grab sampling and core drilling shall be tested for unconfined compressive strength by the Contractor and Engineer, respectively. The Contractor shall fill each core hole with neat cement grout (w/c = 1:1) immediately upon the completion of coring in that hole, using tremie grout methods, unless otherwise directed by the Engineer. The Engineer will select 5 – 10 specimens of the core retrieved from each drill hole for unconfined compressive strength testing. The retrieval of wet grab samples and the core drilling into the trial SCCWs shall be considered part of the Contractor's QA/QC program.

3.2.5. For all soil-cement trial installations drilled, core specimens shall be subject to the acceptance criteria listed in Paragraph 3.7. The Contractor shall not commence any production soil-cement mixing until the Engineer has confirmed that the trial installation has met the acceptance criteria. If the trial installation does not meet the acceptance criteria (as applied to 6 cored SCCWs), then either the Contractor may propose drilling core holes in another 6 SCCWs, which the Engineer will select, or the Contractor may propose to make a second, totally separate trial installation of (*Fill in number*) SCCWs in an area within 100 ft. of the first. If the Contractor chooses to install a second trial installation, it shall be made using procedures and grout mix different from the first trial installation attempt that did not meet acceptance criteria. No less than 10 working days before intended start of this second trial installation, the Contractor shall submit to the Engineer for review and acceptance an explanation for the failure of the first trial installation to meet acceptance criteria and a complete description of the proposed alternate mix design which shall

be designed to achieve at least 30% greater cement factor than did the first attempted trial installation. Once the Engineer has accepted the proposal for the second trial SCCW installation, the Contractor shall within 15 days commence the second trial and carry out the same number of tests as required for the initial trial installation. If the Contractor's second trial installation of soil-cement mixing is determined by the Engineer to also not satisfy the acceptance criteria, which would require that all tested columns (minimum 6 cored columns) be in compliance with the requirements of paragraph 3.7, then this shall constitute sufficient grounds for the Engineer to place the Contractor in default.

3.3. Construction Methods

3.3.1 The Contractor shall furnish all labor, equipment, and materials necessary to conduct all SCCW stabilization operations. The same mixing machine used in the trial installation shall be used throughout the production mixing as the primary machine, in use more than any other brought to the site. Other machines of identical composition and operation may be used. Other different mixing machines may also be used, but each different machine must first successfully complete a trial SCCW installation, as required in Paragraph 3.2. If at any time, the Contractor's equipment, methods, or materials do not produce a satisfactory SCCW, the Contractor shall alter its equipment, methods, or materials as necessary to make soil-cement that meets the acceptance criteria and otherwise complies with the Contract Documents, at no additional expense to the Department.

3.3.2 The Contractor shall begin the production soil-cement columns mixing using the same procedures and grout mixture that was proposed for and determined to meet SCCW acceptability criteria in the trial installations. The Contractor shall not deviate from these procedures for at least the first full 6 weeks or 12,000 cu. yd of soil-cement mixing per rig (whichever is greater), unless it is determined or suspected that the installed SCCW will not meet acceptance criteria. Variations may be proposed after the initial 6-week period if the Contractor can demonstrate with calculations and trial data that the revised production methods would provide soil-cement that will meet

all of the acceptance criteria. The Contractor shall continuously monitor the soil-cement mixing work, and if necessary, the Contractor shall propose modification to the work area layout and other aspects of the installations, including cement factor, to achieve compliance with contract requirements and acceptance criteria, all subject to the Engineer's review and approval.

3.3.3. Relative to the overall site requirements and preparatory work prior to soil-cement mixing, the Contractor shall perform the following:

a. All survey layouts and utility clearances associated with the SCCW stabilization and coordination with all local, state, and federal agencies having jurisdiction, and all utility companies having facilities that could be impacted by the SCCW construction.

b. Clear and grub the site in accordance with the Contract requirements. Limit grubbing to that needed to remove tree stumps and large roots. Do not remove general root mat. Fill in holes left by stump and root extraction with working mat fill and grade to provide level working surface.

3.3.4. The Contractor shall design, construct, and maintain stable working surfaces for all SCCW stabilization operations.

a. Contractor shall provide a minimum (Fill in if a working mat fill or sand blanket is required) throughout the SCCW work areas, per the Contract Drawings requirements.

b. Contractor shall provide a level working surface of adequate width for efficient SCCW installation. The Contractor shall provide timber crane mats of sufficient width and length to form a stable working platform for the mixing equipment and other support equipment.

3.3.5. The Contractor shall mobilize and maintain a sufficient number of personnel and equipment to meet the schedule completion dates and other milestones. Within 14 calendar days after receiving the Engineer's notification of acceptance of the laboratory 'bench-scale'

mix design testing results, and the Engineer's authorization to commence with initial trial SCCW installations, the Contractor shall mobilize to the site the equipment, labor, and materials required to commence and expeditiously perform the initial trial SCCW installations at each of the three trial areas designated by the Engineer. The Contractor shall complete the trial SCCW installations within 28 days of Engineer's acceptance of laboratory 'bench-scale' mix design test results, and complete all core drilling at 28 days after each column selected for coring was installed.

3.3.6. The Contractor shall coordinate all SCCW mixing operations with all other aspects of the Work, including that of the adjacent PV Drain installation.

3.3.7. The Contractor shall perform SCCW stabilization in rows of overlapping columns, where panels are required, in such sequence that does not create obstructions or hindrances to subsequent columns. Sequence of overlapping SCCWs shall provide for secondary, in-fill columns to be installed after the two adjacent columns, as indicated on Contract Drawings, in a schedule that does not cause instability of previously installed columns. The Contractor shall take all necessary measures to overcome difficult augering or excavation hindrance created by SCCW during earlier phases of the Work, at no additional cost to the Department. If the installation of secondary columns does not in the opinion of the Engineer appear to maintain the required overlap with the adjacent primary elements, then the Engineer may direct that the Contractor halt all SCCW work until the Contractor can verify that the overlap has been maintained for full depth of the columns.

3.3.8. Cement Grout Preparation

a. The dry materials shall be fed to the mixers for agitation and shearing. The mixing ratio of the grout shall be controlled by measuring the weight of grout components using automatic batch scales in the mixing plant.

b. Grout shall not be held for more than two hours.

3.4 Installation of Soil-Cement

3.4.1. The Contractor shall design a soil-cement installation sequence layout to achieve the plan area coverage (in single columns or panels of overlapped columns) as designated on the Drawings, to the required elevations and limits.

3.4.2. Soil-cement mixing shall be performed using the cement factor, grout water/cement ratio, and other installation conditions (including penetration and withdrawal rate, grout flow rate, rotation speed, re-penetration depths) that were applied at the successful trial installations (having met the acceptance criteria), and which were accepted by the Engineer. The Contractor shall perform the soil-cement mixing to produce the required stability panels for buttresses using interconnected and overlapped soil-cement columns, by installing separated outer columns (primary), with each secondary column installed between two primary columns.

3.4.3. Achieve the single column/element plan area coverage required for areas of independent columns using a general pattern of equilateral triangles to achieve the spacing criteria indicated on the Contract Drawings. Install all soil-cement columns over the required elevations and limits and to the required compressive strengths specified herein.

3.4.4. The installation of each column shall be continuous. If an interruption of more than 2 hours occurs, the column shall be remixed for the entire height of the column, using fresh cement grout as though there had not been any cement grout installed, or the column may be abandoned, at no cost or schedule impact to the Department. The Contractor shall install additional columns if the interrupted column cannot be acceptably remixed.

3.4.5. Deviations from plan locations at the ground surface shall not exceed 4 inches. Deviation of column from vertical inclination shall not exceed 1 percent. Maintain rig mast and leads in vertical position, and monitor continuously throughout each installation.

3.4.6. Soil-cement stabilization shall be performed beginning at working ground surface level and shall penetrate generally to the depths shown

on the Design Summary Table and illustrated in the Typical Section Drawings on the Plans.

a. The Contractor shall for each SCCW element as it is being completed compare the field-encountered elevation to that indicated on the Design Summary Tables of the Contract Drawings, and immediately bring any discrepancy of more than 0.5 foot to the Engineer's attention. The Contractor shall not begin final withdrawal of mixing equipment from the soil-cement column until the Engineer has approved the depth of the soil-cement column.

b. If multiple shaft equipment with varying mixing shaft lengths is used, the shortest shaft shall extend into the designated depth to provide the minimum treatment depth shown on the Plans. The Contractor will not be compensated for any portion of any SCCW element that does not penetrate the required depth approved by the Engineer.

3.4.7. All soil-cement mixing shall be performed by injecting cement grout into the augered soils in a manner as proposed by the Contractor in the submittal, subject to the Engineer's review and approval. Grout volumes injected shall be sufficient throughout each column to provide at least the minimum required compressive strength throughout the full diameter and length of the column.

3.4.8. In the event geotechnical instrumentation or other types of instrumentation indicate the soil-cement mixing is causing lateral column movements that exceed 1/2 inch, the Contractor shall take immediate action to modify construction sequence or procedures to stop continuation of movements and prevent future movements. The Contractor shall if necessary, or as directed by the Engineer, stop all soil-cement stabilization operations in order to implement the proposed action(s). Soil-cement stabilization operations shall not resume until such corrective actions prove satisfactory to the Engineer.

3.4.9. Immediately after completing a soil-cement column, the Contractor shall install protective covers to prevent persons from falling or stepping into the unhardened soil-cement column.

3.4.10. During the course of soil-cement stabilization, return/spoil material shall not be dumped into or otherwise be allowed to enter the soil-cement column. Construct a silt and spoil containment system to channel spoil to the temporary holding pit in such a direction and manner to keep the spoils away from the site perimeter, and out of the traveled paths. Some portion of the spoils may be used later as embankment fill, providing it is not contaminated and hardens to required degree, as noted in paragraph 1.3.6.

3.4.11. At any soil-cement column which exhibits partial or total instability, or collapses as a result of mechanical failure of any equipment; inadequacy of cement, water supplies, cement grout; improper drilling, injection or mixing procedures; or other cause, the Contractor shall halt soil-cement stabilization, and backfill to ground surface with cement grout. After the backfill has attained sufficient strength to stabilize the ground, complete the required installation by redrilling from ground surface, at no additional expense to the Department. The Engineer will evaluate the potential impacts of the instability and may require one or more additional redrilled columns at overlapping or adjacent locations as determined by the Engineer, at no additional expense to the Department.

3.5 Obstructions

3.5.1. SCCW stabilization is to be carried out in areas known to contain obstructions. Known obstructions, such as (*Fill in details)* logs, stumps, rubble, pavements, or utilities, which do not become evident until the soil-cement mixing process has begun, may be encountered and will be considered unknown obstructions. The manner in which these are treated or removed shall be determined on a case-by-case basis.

3.5.2. Where unknown obstructions are encountered during the operation of (or related to) soil-cement installation, the Contractor shall be prepared to remove or encapsulate the obstruction. When an obstruction is encountered, the Contractor shall notify the Engineer in writing by the end of that day of such encounter, and shall provide all pertinent information relating to depth, plan location coordinates, and expected

extent of the obstruction within 24 hours. Within one day of such notification, the Contractor shall submit a proposal for the Engineer review that delineates the Contractor's proposed means and methods to overcome the obstruction, including equipment and labor time estimated for this operation. Such construction to remove an unanticipated obstruction shall only be performed with the written authorization of the Engineer, and shall be compensated as indicated in paragraph 4.3 herein.

3.6 Collection of Soil-Cement Return

3.6.1. At all times during and at completion of soil-cement stabilization operations, the site shall be maintained clear of all debris. Soil-cement return and spoil material shall be piped or channeled to holding ponds or other retention structures within the work area. The Contractor shall separate for specific stockpile locations those hardened spoil materials that are proposed for reuse as "bridging fill." The Contractor shall regularly dispose of all waste materials (including those spoils that are not to be reused as hardened fill materials) in a manner acceptable to the Department, the Department of Environmental Quality, and all other agencies having jurisdiction. Such collection and disposal methods shall be thoroughly explained and shown in the Contractor's initial fieldwork submittals to the Engineer prior to the start of soil-cement stabilization operations.

3.6.2. The Contractor shall take all necessary precautions and implement measures to prevent any soil-cement return, other spoil material or stockpiled materials from entering storm drain structures, drainage courses, other utility lines, or from leaving the site via surface runoff. The Contractor shall prevent soil-cement return, fluid, ponded spoil material, or stockpiled solidified materials from migrating into any water body.

3.6.3. In the event soil-cement return, spoil material or stockpiled materials enter storm drain structures, drainage courses, or other utilities, including, but not limited to, surface water bodies beyond site limits of soil-cement mixing operations, the Contractor shall collect and remove all of these materials, and perform all other required/necessary remediation that may be directed by the Engineer or responsible

environmental agency, at no additional cost or schedule impact to the Department.

3.6.4. The Contractor shall conduct all soil-cement operations to conform to sedimentation and turbidity control requirements of federal, state, and local agencies having jurisdiction over the Work.

3.7 Acceptance Criteria

The completed SCCWs or panels shall meet all of the following conditions to the satisfaction of the Engineer for an element to be considered acceptable and compliant.

3.7.1. The horizontal and vertical alignment shall be within Contract requirements noted in this specification.

3.7.2. The top of the columns or panels shall be placed (*to the top elevations)* in the Plans.

3.7.3. The minimum diameter and spacing of each SCCW shall be within Contract requirements, and as required by the Engineer.

3.7.4. The bottom of each element shall be at least as deep as that shown on the plans, and shall have been shown to the satisfaction of the Engineer in the field (*If terminated in a dense deposit detail criteria, such as using **augering** refusal defined by both **augering** rate, and either resistance in amperage or increased hydraulic pressure, depending on means used for turning the mixing tools).*

3.7.5. Conditions of Soil-Cement, Based on Retrieved Core Samples: The acceptability of soil-cement columns will be assessed on the basis of the core conditions listed below. If the core of one column is determined to not meet each of these acceptance criteria, then the SCCW element shall be considered non-compliant, and the Contractor shall install supplemental columns at no additional cost or schedule impact to the Department. With the finding of a non-compliant core, all other soil-cement columns installed since the previous successfully cored column that was tested and determined to meet these acceptability criteria shall be considered also non-compliant. Such

designation as 'non-compliance by association' can only be removed by performing additional core hole drilling in at least 10% of the columns, but not in fewer than 2 columns, at no additional cost or schedule impact to the Department. Otherwise, the Contractor shall be required to install additional soil-cement columns to supplement all columns designated as 'non-compliant by association', at no additional cost or schedule impact to the Department.

a. Recovery: The minimum core recovery shall be 85%, determined for each individual column location cored, based on the full length of coring. Also, each 5-ft. long core run in the depth range of the clay soils stratum shall have at least 80% recovery. If successive core holes each fail to retrieve core of soil-cement in the same elevation horizon(s) within the alluvial soils stratum, then the columns drilled and others installed between these columns shall be considered non-compliant. See also Paragraph 1.4.7.e. and 1.4.7.f. for additional criteria that require drilling of additional core holes.

b. Uniformity: The soil-cement prepared to compose columns or panels shall be uniformly mixed and shall not include large and/or aggregated lumps of unimproved or weakly cemented soil. Weakly cemented soil (weak soil-cement) shall generally be determined by unconfined compression tests on core specimens. The Engineer will also use a pocket penetrometer to probe the core to determine both variability of strength and those zones where strength is indicated to be less than *(Fill in contract unconfined compression strength)*, since it is considered not practicable to test all of the recovered soil-cement core (in some cases due to its insufficient integrity for use in unconfined compression because of breaks, fractures, voids, or other deficiencies) and for efficiency of core characterization to limit the numbers of tests. Over any SCCW core, if either of the following conditions occurs, then the cored columns shall be designated non-compliant:

    1. The zones of unimproved or weakly-cemented soils (identified as pocket penetrometer readings indicating less than *(Fill strength from b above)* reading on the high-capacity pocket penetrometer) greater than 2.0-inch size amount to more than

10% of the total volume of the core, or more than 10% of the core surface area.

2. Or, any individual or aggregation of lumps of unimproved or weakly-cemented soil is larger than 5 inches in greatest dimension.

   The Contractor shall propose to install additional SCCWs to supplement non-compliant columns at no additional cost or schedule impact to the Department.

c. Strength: The soil-cement specimens tested shall achieve minimum and average 28-day in-situ unconfined compressive strength listed below, which will be determined on the basis of running capped average of unconfined compressive strengths on the core recovered from each separate column, with any test having strength greater than (*Fill in 25% greater strength than the contract unconfined compressive strength)* being included. If either the Running Capped Average (RCA) for a single column is less than (*Fill in contract unconfined compression strength)* or any soil-cement specimen tested has less than *(Fill in typically a strength about 50% less than the contract unconfined compressive strength*), then the cored soil-cement column shall have not met the acceptance criteria and shall be designated to be non-compliant. The Contractor shall propose to install additional SCCWs to supplement non-compliant columns at no additional cost or schedule impact to the Department.

d. Inconsistency of Production of Acceptable Soil-Cement: If non-compliance is determined to recur in certain depth intervals either due to the lack of core recovery, soil-cement core non-uniformity in visual/manual examination, or from the results of the unconfined compressive strength tests (which may indicate difficulty in mixing certain soil layers), then the Contractor shall immediately at the direction of the Engineer stop further soil-cement column mixing operations. The Contractor shall propose remedial measures to both supplement installed columns considered to be non-compliant and to modify future soil-cement mixing operations to overcome the conditions that have caused soil-cement to be non-compliant,

which shall all be at no additional cost or schedule impact to the Department.

3.7.6. Wet Grab Samples: The unconfined compressive strength of soil-cement determined from wet grab samples, if taken, will be used only as a preliminary indicator of apparent soil-cement columns' unconfined compressive strength, and not as an accurate measure of in-situ strength. The examinations and compression tests made ***on core samples shall be the determining measure of soil-cement acceptability or non-compliance.***

3.8 Fill Placement Subsequent to Soil-Cement Installation

The Contractor shall not place any fill materials or impose any other loadings over the columns in any given area for a minimum of 35 days after installation of all of the soil-cement columns in that area. This waiting period is to allow the newly installed soil-cement enough time to gain strength necessary for supporting fill and equipment loads. The Contractor shall not place fill in any area until all QA/QC coring required by the Engineer has been completed in the area and accepted by the Engineer.

4.0 MEASUREMENT AND PAYMENT

4.1 General

The contract unit price for each item shall constitute full compensation for furnishing all plant, labor, equipment, materials, and supplies, special compression testing by a laboratory, and all aspects of the QC/QA program, except as otherwise specified, and for performing all operations required for the satisfactory completion of all work included under the items as specified herein, shown on the Plans, or directed. All measurements for payment shall be made in the presence of the Engineer. Payment will not be made for SCCW work not installed to the specified depth or which does not meet acceptability criteria, nor will payment be made for any soil-cement element for which satisfactory records are not furnished to the Engineer, as determined by the Engineer.

4.2 Soil-Cement Columns (SCCWs)

SCCWs shall be installed in accordance with the plans and specifications. Satisfactorily installed columns, including satisfactorily installed complaint trial columns, shall be included in payment calculations. Such payment calculations shall be made per cubic yard, to the nearest tenth of a cubic yard for only the minimum percent area coverage ***and end junction columns/elements*** of soil-cement shown on the Drawings, or approved by the Engineer, except that trial columns installed outside of the area shown on the drawings and which meet the acceptance criteria and are approved by the Engineer will also be included in payment calculations, but only on the basis of 3-foot-diameter column elements. The volume of columns added by direction of the Engineer to accommodate future Construction and which meet the acceptance criteria and are accepted by the Engineer will also be included in payment calculations. The volumes shall be determined by multiplying the plan area (which shall not exceed the specified minimum area coverage indicated in the Design Summary Tables (*Add if applicable plus end junction column areas between stability buttresses, both computed on the basis of 3-ft-diameter column elements)* times the actual depth of soil-cement installation, from the top elevation to the agreed bottom elevation. The volume of trial columns shall use the number of columns that meet the acceptance criteria as shown on the submitted shop drawings that are approved by the Engineer. No measurement will be made for any SCCW element that does not reach the required depth, or that does not satisfy the acceptance criteria, or which is designated non-compliant. Areas of overlapping and over-drilling will not be measured more than once.

4.2.1. No separate payment or schedule impact allowance will be made for additional quantities of soil-cement installed to overcome obstructions. ***However, installed volume of soil-cement of any column that cannot be completed due to encountering an unknown obstruction shall be included in the measurement of the SCCW, and will be paid for at the contract unit price for the SCCW.***

4.2.2. Soil-cement columns installed to replace or supplement non-compliant soil-cement or that are outside the limits of soil-cement stabilization shown on the Drawings (except for approved trial columns) will not be measured for payment without the approval of the Engineer.

4.2.3. No separate payment or schedule impact allowance will be made for the Contractor's Quality-Assurance/Quality Control Program, including, but not limited to, obtaining wet grab samples and making and testing cylinders, core drilling, and specimen identification; preserving; transporting; and core hole grouting, which will be considered incidental to the Work.

4.2.4. No separate payment or schedule impact allowance will be made for making test borings or excavating test pits to retrieve soil samples or for soil classification testing; sheeting and shoring, if required; mobilization, demobilization; ground treatment or providing of timber crane mats, Geosynthetics, granular fill, or the like to make the site suitable to support soil-cement mixing equipment, or core drilling of additional holes as required by the Engineer to assess extent of any soil-cement designated as non-compliant.

4.2.5. No separate payment or schedule impact allowance will be made for any aspect of removal of known obstructions, or for idle or waiting time, or for mobilization/demobilization for any unknown obstruction removal work.

4.2.6. Trial SCCWs provided that are outside of production locations ***that meet the requirements of this specification for production SCCWs, and are approved by the Engineer*** will be measured and paid for as production SCCW work.

4.2.7. No separate payment or schedule impact allowance will be made for mobilization or demobilization of any soil-cement mixing equipment. The cost of such mobilization/demobilization shall be paid for under the overall contract item for the entire project.

Soil cement column work (SCCW) will be measured and paid for at the contract unit price per cubic yard, which price shall be full compensation for the cost of installing the full volume of the SCCW complete-in-place to the specified depth. The contract unit price shall include all costs for any alterations to the equipment and methods of installation and working surface geometry in order to produce the required end result in accordance with the plans and specifications; it

will also include the cost for providing all tools, materials, labor, record information, and all other costs necessary to complete the required work.

4.3 Obstruction Removal

Removal of an unknown obstruction, authorized in writing by the Engineer and performed to the satisfaction of the Engineer, will be measured as time actually spent by the Contractor in the removal operation. The measure of an hour of work shall include the entire crew, material used, equipment utilized, equipment mobilization/demobilization, and any incidentals to remove the obstruction.

Removal of an unknown obstruction, authorized by the Engineer and performed to the satisfaction of the Engineer, will be reimbursed as 'force account work' for time and material. That is, on the basis of hours of activity applied to each particular obstruction removal operation by the Engineer for each unknown obstruction removal operation. The measure of an hour of work shall include the entire crew, all materials used, and all equipment utilized, equipment mobilization/demobilization and any incidentals required to remove the obstruction, and shall only be for time during active removal activities.

4.4 Coring of Additional SCCWs

The core drilling of SCCWs that is not included within the Contractor's QA/QC program and that is required by the Engineer to make supplemental investigations will be measured and paid for by the linear foot of coring, beginning at the ground surface at the time of coring and proceeding downward to the bottom of the core hole, but not deeper than the depth of the installed soil-cement column that is being drilled. The cost will include, but not be limited to, all materials and equipment necessary to retrieve the core specimens, including the specified core barrel and related tools, supply of water, core boxes, and related materials to provide moisture retention, person to log the core specimens, and transporting and preserving core specimens. Measurement will be made to the nearest 0.1 ft. No measurement will be made for depths of core holes that must be redrilled due to collapse or other problems. No measurement of payment will be made for any part of core holes that are not completed satisfactorily to the required depth.

Payment will be made under:

Pay Item	**Pay Unit**
Individual Soil-Cement Columns Wet	Cubic Yard
Overlapped Soil Cement Columns Wet	Cubic Yard
Coring of SCCWs	Linear Foot
Obstruction Removal	Hour

5.2 CONSTRUCTION MONITORING

a. Pre-Production or Pre-Construction Testing

As previously outlined, the improved properties of the soil mixed mass are a function of the variability of the in-situ soil, the reagents used, the characteristics of the mixing equipment, and the rate of penetration and withdrawal. Many of these variables cannot be studied adequately during design, since the equipment ultimately used is unknown and only limited subsurface samples are typically available.

Therefore, a pre-production program must be undertaken prior to full production. The primary purpose of the program is to establish production parameters that will be the procedures and materials to be used during the production phase.

As part of this program, trial soil mixed panels or columns are constructed, monitored for construction methods, alignment tolerances, and sufficient samples recovered to establish that the required engineering characteristics for the mixed soils have been achieved. As part of this work, the Contractor is typically required to submit:

- Qualifications for personnel for soil mixing.
- Equipment, procedures, and materials to be used.
- Spoil containment methods and structures.
- Sequence and time schedules for all operations.
- Mix design calculations, including reagent type, quantity, water reagent ratio, if wet mix, and estimated 28 and/or 56-day compressive strength, or other required engineering properties.
- Results of any bench scale or mix design laboratory tests.
- 28-day compressive strengths from the first week of installations, or other in-situ measurements of strength specified.
- Column diameter achieved, injection pressure and rates, withdrawal rates, rotation speeds, volume ratios, and reagent factor.

- Methods and procedures to overcome obstructions.
- The details, frequency, and documentation format for the Quality Assurance/ Quality Control Program.
- Measurements that demonstrate that the required alignment can be met.
- Developed plan for the installation of any required instrumentation or performance of in-situ load testing or observation wells, as required.

b. Monitoring

Specifications for both wet and dry methods include extensive sections devoted to monitoring and quality control. It should be noted that as both technologies have not been extensively used to date, monitoring and Quality Control methods are still evolving.

Items to be checked during construction include freshness of stabilizers, particle sizes, location of columns, eccentricity, length of columns, quantity of reagent utilized, withdrawal/mixing rate. On large projects (>1000 columns) or critical applications, it is advisable to check the bearing capacity and the creep limit with full-scale test columns at the project site.

Verification of in-situ strength by testing methods chosen for each project is an essential aspect of QA/QC programs. Table 6 summarizes the elements of QA/QC testing to be implemented on each project.

Table 6. QC/QA testing items for soil mix columns.[1]

Subject		Test Items	Instrument	Frequency
QC	Quantity of cement	Total weight	Delivery record	Daily
	Cement grout	Specific gravity	Mud balance	Each batch
	Quantity of grout	Volume	Flow meter	Each column
	Mix condition	Drilling speed Rotation speed Wet sampling	Record Record Sampling tool	Each column Each column 1 to 2 times/day
	Length	Column length	Drill stem	Each column
QA	Diameter		Tool diameter	Daily
	Continuity	Core drilling	Visual	Random sampling
	Uniformity	Core drilling	Visual	Random sampling
	Strength	Core drilling	Unconfined compression test	Random sampling

It is clear from the descriptions of the different techniques summarized in Table 3 that installation process control is a critical element in assuring the quality of the treated soil. Computerized systems provide a historical record of the operation and can provide confirmation that the performance requirements have been met during construction. Bruce has identified process controls generally, as follows:[1]

Level 1

Batching and injection parameters for the slurry (or dry binder) are monitored by simple instrumentation and are displayed on digital or analog gages for field personnel to view. Spot checks are made manually on slurry fluid properties, e.g., density (by Baroid Mud Balance), fluidity (by Marsh Cone), and so on. Batching and injection parameters are controlled by computer, and are preset to provide a pre-selected volume ratio and cement factor, which is closely related to shaft penetration rate. In turn, these data are automatically recorded and displayed, with visual confirmation to the rig operator that they are within the pre-selected parametric range. If not, manual corrections may be made. Full electronic records are made for each column of all salient drilling and injection parameters. Spot checks are made of fluid slurry properties.

At the end of each working day, the system should be able to print out a daily report and production total report that includes the number of columns, their length, and the amount of reagent used. A centralized control system thus helps produce high quality soil mix columns, simplifies the work of preparing various reports, and saves time and labor. Level 1 should be considered the minimum level consistent with small projects.

Level 2

The highest level of computer control and display is provided. It features a microprocessor that senses, at short time intervals, rpm, penetration rate, torque, thrust, slurry density, pressure, amperage, and rate. The computer reacts to changing ground conditions and automatically adjusts injection parameters to maintain specific treated soil parameters for each stratum. Rotation is stopped automatically if these projected treated soil parameters are unlikely to meet preset limits. The drill operator has a touch screen control system. This level is also characterized by full continuous records for each column installed.

Summary

Examination of the process control records and soil mix property verification is one of the principal bases of construction monitoring.

CHAPTER 6
COST DATA

Wet mixing cost data is available from more than a dozen projects completed in the last decade in the United States. By contrast, only one dry mix lime-cement project has been implemented under a design-build contract. Therefore, cost information cannot be independently confirmed.

6.1 BUDGET COSTS FOR DEEP MIXING METHODS

Given the large cranes required to support the multiple mixing augers, coupled with the specially designed on-site batcher, the current minimum cost for mobilization/demobilization for deep soil mixing using wet methods is in the vicinity of $100,000. In addition, the cost per cubic meter of mixed soil will vary depending on the normal project variables, e.g., labor situation, size of the project, depth and type of in-situ soil to be treated, weather conditions, etc. A budget cost of $80 – $150 per cubic meter ($70 – $125/cu.yd.) of volume treated is a reasonable range for large complex projects, not including mobilization. Smaller projects or excavation support projects should be budgeted at $65 – $100 per cubic meter ($55 – $84/cu.yd.), and with a somewhat smaller mobilization cost.

The following additional costs should be added

- mobilization/demobilization
- any pre-excavation required for obstruction removal and installation of guide wall
- removal and disposal of spoil
- the in-place cost of any reinforcing elements, including waler, tieback anchors, etc., for retaining wall applications

6.2 BUDGET COSTS FOR LIME-CEMENT COLUMNS

It is reported that costs in Scandinavia are in the range of $10 – $20 per linear meter ($33 – $65/lft.) of lime-cement columns, with United States costs projected higher. The reported cost of the first lime-cement column project at I-15 in Salt Lake City was $30 lineal meter ($100/lft.). On average, this would suggest a cost of $60 per cubic meter ($50/cu.yd.). Mobilization costs are somewhat lower than with wet mix equipment. Costs are material sensitive, with approximately 40% of the total cost attributed to reagent materials. For

maximum construction efficiency, the columns should be in the range of 10 – 20 m (30 – 65 ft.) in length.

6.3 OTHER COSTS

Although the data is sparse, it appears that the Owner's QA program should be budgeted at $2.00 – $4.00 per cubic meter ($1.65 – $3.35/cu.yd.). This cost would include on-site inspection of operations and the cost of testing cores.

CHAPTER 7
CASE HISTORIES

Soil mixing techniques are emerging technologies in the United States. Consequently, local case histories are few. For lime columns, the only United States project to date has been a test program. For lime-cement columns one major project has been constructed in Utah on I-15.

7.1 EXCAVATION SUPPORT USING WET DMM, FOR THIRD TUNNEL PROJECT, BOSTON CENTRAL ARTERY[9]

This project case history describes the construction of an anchored soil mixed wall for a highway application. The project was accomplished as an alternate to conventional groundwater cutoff and excavation support technology. Since this project was completed, extensive application of this technology that was utilized on the Boston Central Artery project.

Background

As part of the Third Harbor Tunnel project, a cut-and-cover tunnel was designed to connect Logan International Airport and the immersed twin steel tube tunnel crossing the Boston Harbor, as shown in Figure 19. The excavation was 1,100 m (3,520 ft.) long, with the depth ranging from 12 – 26 m (40 – 85 ft.). The majority of the cut sections were supported by a soil mix wall to provide a groundwater cut-off and a structural excavation support. Additional lateral support was provided by 2,700 pre-stressed soil anchors. An approximately 915 linear meter long soil mix wall with a total wall area of 37,160 m^2 (400,000 ft^2) was designed and constructed. The soil mixed wall was constructed using 0.9 m (3.0 ft.) diameter augers and mixing paddles that penetrated the soil and adjacent column while injecting a cement/water grout. In every other auger hole, a vertical W21 grade 50 steel beam (soldier pile) was placed into the fluid, treated soil.

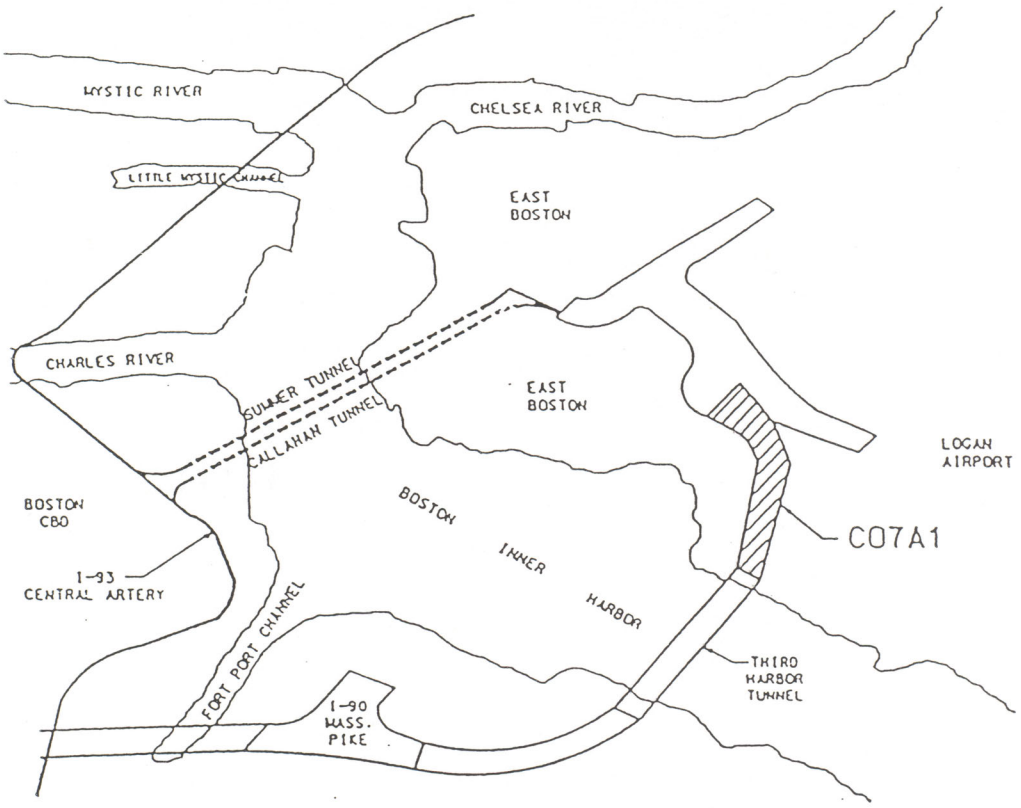

Figure 19. Site Plan for Deep Mix Wall - Boston Central Artery.

Subsurface Profile

The subsurface conditions in the cut-and-cover tunnel were influenced by a complex historical site filling and various past site uses. The subsurface materials in descending order consisted of fill, organic deposits, marine clay, glacial till deposits, and bedrock. Some strata are not prevalent in certain sections of the tunnel alignment. For the purpose of excavation support design, three distinct areas along the tunnel alignment were identified as follows:

- Zone A - Station 135+75 to 146+74. Sandy fill overlying organics and dense glaciomarine deposits. The glaciomarine dominates the anticipated face of excavation through this zone.

- Zone B - Station 146+74 to 156+00. Various deposits of sandy fill, cohesive fill, and man-made debris that filled a trapezoidal ship channel overlying very dense glaciomarine deposits at the approximate tunnel subgrade.

- Zone C - Station 156+00 to 163+72 and ramp E-T Sta 368+10. Fills (both cohesive and granular) overlying organic, marine clay, and glaciomarine. In this section, the marine clay deposit was much thicker than in Zones A and B.

In all areas, the ground elevation was approximately 34 m (112 ft.) and the design groundwater table was at about 33 m (108).

Design Issues

The design of the excavation support addressed the following major issues:

- Support the lateral earth and water pressures, as given by the plans.
- Prevent drawdown of the surrounding groundwater table.
- Provide wall stiffness and corresponding construction quality to result in minimal lateral ground movements and surface settlements.

The project specifications indicated that for three soil zones, A, B, and C, the following shoring systems could be used to meet the constraints:

- Zone A. Predrilled soldier piles and wood lagging with an impermeable cutoff trench placed behind the piling and keyed into the glaciomarine soil prior to any face excavation.
- Zone B. A reinforced concrete diaphragm wall (slurry wall) constructed using the slurry panel method.
- Zone C. Driven interlocking steel sheet piles.

The original design did not include a soil mix wall concept. In accordance with project specifications, however, the contractor was allowed to propose alternative support systems that were able to meet the performance requirements and site constraints.

As a result of technical and cost evaluations and comparisons between the deep soil mixing technique proposed by the contractor and the originally specified techniques, the following alternative approaches were accepted as being both cost effective and able to meet project performance requirements:

- Zone A. As an alternate to the soldier pile and lagging with integral cutoff, a low permeability soil-mix wall was proposed that would be equal to or better than the soldier pile and lagging with integral cutoff.

- Zone B. As an alternate to a concrete slurry wall, a soil mixed wall system was proposed. Equivalent impermeability and adequate vertical rigidity and strength would be provided by the soil mixed wall at a lower cost, as compared to the slurry wall.

- Zone C. As an alternate to driven interlocking sheet piles, a soil mixed wall was proposed that would provide better support due to increased vertical stiffness and far less horizontal permeability.

Phased submittals were produced, corresponding to the anticipated construction schedule, for each general soil zone, since pressure diagrams and other design constraints varied with the ground conditions.

Aside from the numerous wale configurations that accommodated the varying anchor angles and horizontal spacing, the structural design of soldier piles and horizontal wale elements followed typical practice for the structural design of such shoring systems. There are two significant design differences that were implemented for the design of excavation supports.

- *Vertical Support* - Since the construction technique mixes cement slurry directly into the soil, a significant vertical resistance is developed along the entire face of the wall section. A bond value of mixed soil wall to soil was assumed to act over the entire height of the wall to provide vertical resistance of the downward forces applied to the system (anchor forces, self weight, bridge loadings, etc.). In addition, it was assumed that the bond to the steel piles was developed such that the entire base area of treated soil is providing resistance in end bearing.

- *Lagging Design* - Most soldier pile and lagging designs assume that the lagging is acting flexurally. Because of the aspect ratio of the soil-cement material between the beams, a flexural model is inappropriate. Instead, it was assumed that the mixed soil material works as a compression arch between the soldier piles. In addition, lagging adequacy is checked for punching shear (V) at the point of maximum shear along the edge of the pile, and at a point of lesser shear and minimum section at the necking point between the soil mix columns.

Construction Technique

The soil mixed wall was formed by an arrangement of three overlapping mixing tools guided by a lead mounted on a crawler-based machine. Typically, the soil mixed construction process begins by positioning the base machine and augers in conjunction with a steel template. The operator then begins the drilling process and notifies the automated batch plant operator to begin pumping the cement grout to the augers.

The project soil profile generally consisted of 6 – 7 m (20 – 23 ft.) of cohesive and granular fill overlying the native glaciomarine deposits or marine clays, with the exception of the ship channel area (Zone B) where fills reached depths of 20 m (65 ft.). In all cases, the fill material was "pretrenched" along the wall alignment to remove all man-made obstructions, such as granite blocks, steel and wood piles, and concrete rubble. This work was done using a large hydraulic backhoe and, with the exception of the ship channel area, was performed by open cutting and immediate backfilling. The ship channel area was pretrenched under bentonite slurry to stabilize the excavation.

After pretrenching was completed, a 1.5 m (5 ft.) deep guide trench was excavated along the wall alignment to provide containment and access for removal of the excess soil-cement material that develops during the mixing process. The amount of excess “spoil” material is determined by the type of soil being mixed. In the glaciomarine deposits and fill material, the quantity was approximately 30% of the total soil treated. In marine clays, the excess quantities were as high as 50%. A small hydraulic backhoe was used to excavate the trench and the spoil was removed from the trench, on a continuous basis and stockpiled in a pit.

On typical projects, the spoil sets up to a gel consistency within 24 hours and is removed by truck to a project specified dumpsite. In urban settings, the spoil can be loaded directly into trucks for immediate removal, but for this project the material was temporarily stockpiled in order to economize trucking. Upon completion of the initial run of a guide trench, a steel template was set to establish the correct wall alignment and beam placement.

The first panel drilled was a primary panel. Upon completion of the primary panel, the machine was moved along the alignment to the next primary panel, and this panel was installed. The next step was to close the gap between the two primary panels by drilling a secondary panel, which utilizes the previously drilled outer holes of the two adjacent primary panels as a guide (in effect, these holes are redrilled) for alignment and continuity. This was a significant factor that allowed the soil mixing method to provide a continuous wall system with very little opportunity to leave "windows" within the wall. Once the secondary panel was completed, W21 grade 50 beams were set into the outer two holes of the secondary

panel. The process was repeated by installing another primary panel, a secondary panel, then setting two more beams (Figure 20).

On the project, wet bulk samples of the soil-cement mix were taken daily from various depths, and compression tests of molded 76 mm x 150 mm (3.0 x 6 in.) cylinders were performed at 7- and 28-day intervals. Once the wall was cured in-situ (times varied from 7 days to 28 days), the contractor began the excavation process.

Upon excavation and scraping of the wall, double wales were installed prior to the installation of tieback anchors. The wales were prefabricated offsite, generally in 14.5-m (50-ft.) sections and then installed at designated elevations along the wall. Anchor locations and bearing plates were part of the prefabrication so that upon installation, the tieback operation immediately began.

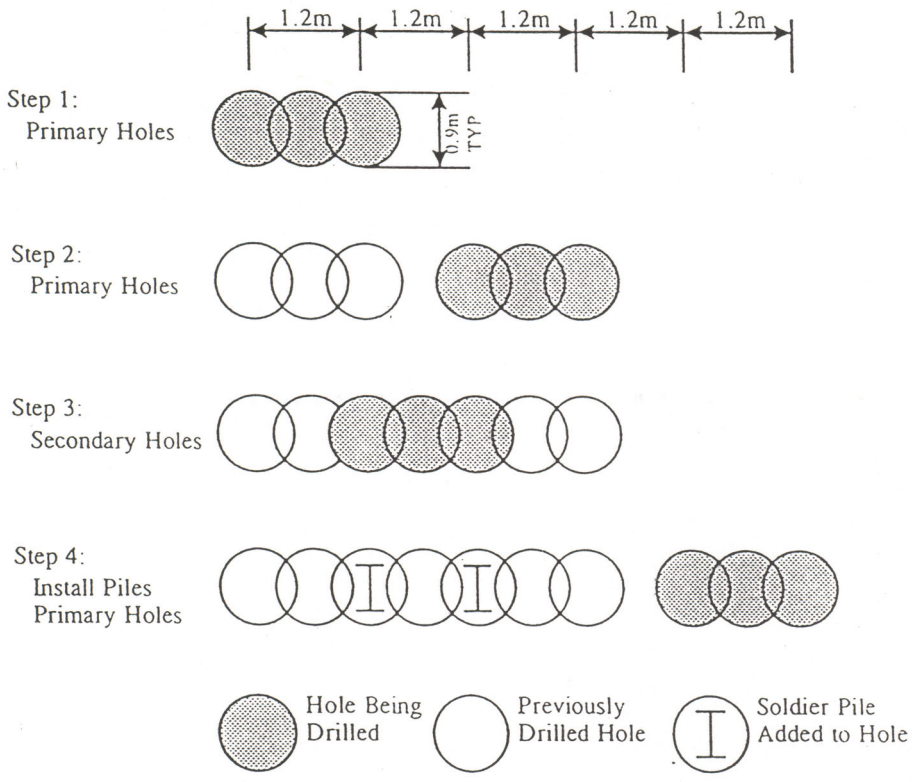

Figure 20. Soil Mixed Wall Panel Installation Sequence.

More than 50% of the ground anchors were post grouted. Typically, all anchors were installed with a post grout tube system, but only those anchors in marine clay or the softer fills actually required post grouting. Any anchor that did not appear to meet movement criteria during stressing was immediately post grouted and then restressed. This factor was significant in maintaining the excavation schedule and in eliminating the need for additional anchors. All anchors were proof tested, and 5% were subjected to a more rigorous cyclic performance test.

Figure 21 shows the nearly completed construction.

Figure 21. Nearly Completed Soil Mix Wall.

Additional Ground Treatment

In Zone C, a section of approximately 152 m (500 ft.), which was underlain by marine clay to a depth of greater than 26 m (85.3 ft.), experienced movement during construction. During the excavation to 13 m (42.6 ft.) for the installation of the third row of anchors, the east wall moved 50 – 76 mm (2.0 – 3.0 in.). A section of approximately 61 m (200 ft.) continued to move 150 – 230 mm (6 – 9 in.) in 15 days. A berm was placed against the wall to prevent further wall movement. It was concluded that further improvement of the marine clay was necessary before construction could proceed.

Numerous options were studied, and remedial measures were selected consisting of

1) installing additional rows of anchors
2) improving the engineering properties of the marine clay

These two remedial measures increased the passive resistance of the toe of the wall and also provided global stability to the excavation support wall.

The ground treatment was performed using deep soil mixing equipment to install buttresses in a pattern, as shown in Figure 22. The buttress wall was extended to the glaciomarine deposits in the eastern area and was extended to the same depth as the bottom of the soldier piles in the western area of the excavation. For load transfer, jet grout columns and soil mixed hammer head columns were placed between the soil mixed wall and the soil mixed buttresses (Figure 22).

One factor that has become apparent as a consequence of the severe weather around Boston Harbor is that the exposed soil-cement, when subjected to freeze-thaw cycles, exhibits deterioration in the form of layers that flake away from the surface. To curb this effect, mix design changes were put in place in order to increase strength, and presumably, freeze-thaw durability.

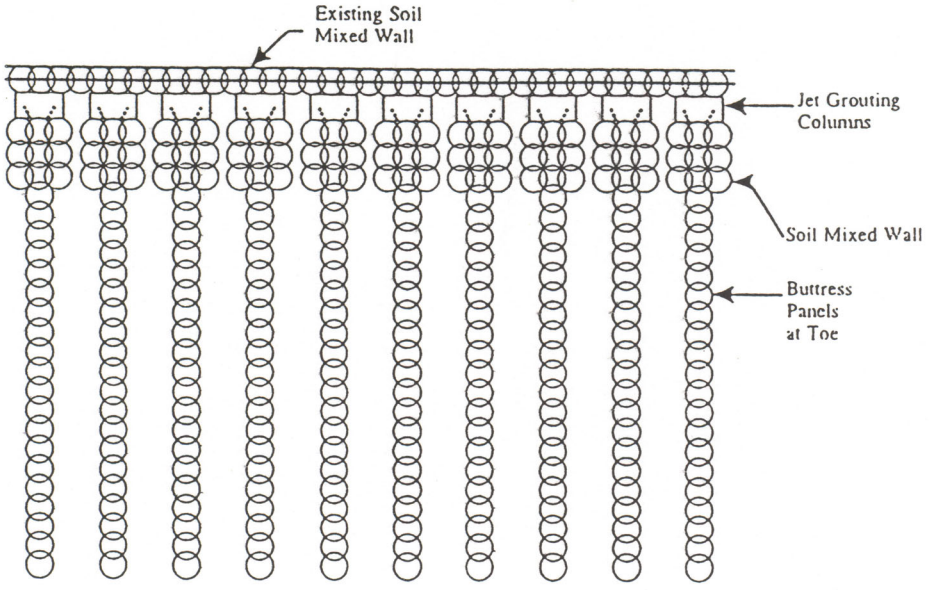

Figure 22. Soil Mixed Wall Buttress for Toe Installation During Excavation.

Summary and Conclusions

The use of an anchored soil mixed excavation support system on this project demonstrated the feasibility of this technology in the United States. From a construction standpoint, the benefits on this project were:

- ability to drill through obstructions
- very low noise and vibration levels
- speed in excavation due to no lost time in placing lagging
- impermeable cutoff providing a dry excavation
- easily accommodated varying anchor and wale placement
- good verticality control
- ability to drill to deep depths

From a design standpoint, the use of deep soil mixing on this project demonstrates that

- 24 m (79 ft.) deep excavations may be made and secured with prestressed ground anchors
- ground deformations can be controlled and are relatively small
- economical designs are feasible due to the ability to develop significant vertical resistance
- a wide range of vertical soldier pile sizes may be selected
- significant lateral earth pressures may be resisted by the stiff arching action of the soil mix between the piles

7.2 EMBANKMENT STABILIZATION/SETTLEMENT REDUCTION, I-15, SALT LAKE CITY

The following case history describes the first highway related application of dry mix lime-cement columns in the United States at I-15, Salt Lake City.[5]

Dry mix LCC stabilization was used to support a mechanically stabilized earth wall (MSE wall) SS-01 located along westbound I-80 between the bridge crossing the UTA Light Rail corridor (200 West) and 300 West. The MSE wall heights ranged between 10 – 12.5m (33 – 41 ft.). A surcharge of as much as 3 m (10 ft.) was required above the final wall elevation in order to pre-compress the soils beneath the tips of the columns so that the differential settlement criteria established for the roadway could be met.

In order to provide the stability required to support these heights of fill on the weak soils of glacial Lake Bonneville, columns about 22 m (72.2 ft.) long and 800 mm (32 in.) and 600 mm (24 in.) in diameter were used.

Subsurface Conditions

Salt Lake City is founded, primarily, on the weak soil deposits of glacial Lake Bonneville. The subsurface conditions at this location are characterized by up to 12 m (40 ft.) of clay with overconsolidation ratios (OCR) of 1.2 to 1.5 below a 3 m (10 ft.) crust. Water content ranges from 30 – 45% percent and P.I.'s on the order of 20. Below this upper clay a 1 – 2 m (3.3 – 6.6 ft.) dense sand layer was encountered, which was typically underlain by interbedded sand and clay layers to great depth. Groundwater was encountered at about 3 m (10 ft.) below grade and artesian pressure from a depth of 18 m (59 ft.) was observed.

Laboratory Testing

Laboratory unconfined compression testing on laboratory-prepared samples to select the appropriate lime-cement mixture were made prior to bid and after the award of the contract. The post-award testing results are shown in Figure 23.

Based on the results of the testing and the design strength required, a mixture of 85% cement and 15% lime, 125 kg/m^3 (7.8 lb/ft^3), was selected for use at I-15.

Design

The MSE wall was supported by panels of overlapping 800 mm (32 in.) columns approximately 22 m (72 ft.) long for the full width of the wall (approximately 0.7H), plus 2 m (6.6 ft.) in front and 2 m (6.6 ft.) behind the wall. The panels were spaced 2 m (6.6 ft.) center to center (2-½ diameters clear) and had two cross-connections around the centroid of loading of the MSE wall.

The retained fill behind the MSE wall was supported on 600 mm (24 in.) columns installed in a rectangular pattern between 1 m and 1.2 m (3.3 ft. and 4 ft.) apart (depending on the height of the fill) and extending beyond the edge of the surcharge loading. In order to reduce differential settlements, a transition zone composed of three gradually shortened columns was installed beneath the back slope of the fill.

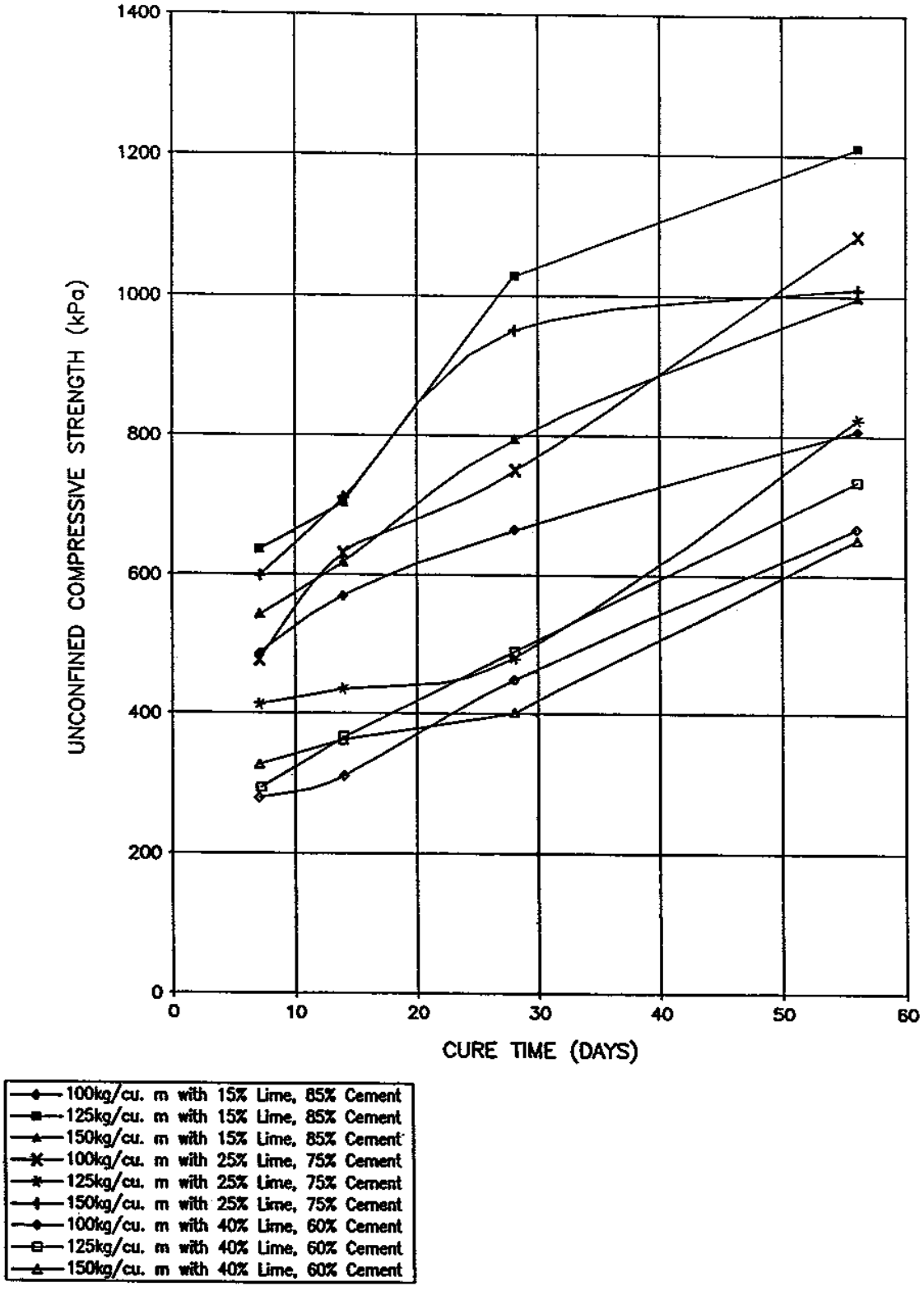

Figure 23. Unconfined Compressive Strength vs. Time.

The 800 mm (31.5 in.) diameter columns were designed for a compressive strength of 600 kPa (12.5 ksf) (undrained shear strength of 300 kPa (6.3 ksf)). The 600-mm (24-in.) diameter columns, where better mixing was expected because of their smaller diameter, were designed for a compressive strength of 800 kPa (16.7 ksf). The undrained shear strength of the untreated Lake Bonneville clay at a point just below the desiccated upper crust was about 45 kPa (940 psf).

Performance Data

Only limited data have been published to date on this project. Figure 24 indicates settlement measured at two typical inclinometer locations for the first five months.

Note that both inclinometers were indicating the same settlements with a maximum settlement after five months of about 100 mm (4 in.). Total settlement of the stabilized ground under the applied wall loads was estimated during design as between about 200 – 300 mm (8 – 12 in.). Projections of the settlements measured to date indicate that total settlements are likely to be less than 150 mm (6 in.). The settlement of the wall if wick drains had been used to permit its construction was estimated as about 1300 mm (50 in.).

The data in Figure 25 are of special interest. The lower half of the Figure shows the stress measured by load cells as the fill was placed. Two load cells were installed, one directly on the lime-cement columns forming a panel, and the other on the untreated soil between two panels.

The stress measurements indicate that the vertical stress applied by the fill of between about 200 – 250 kPa (4.2 – 5.2 ksf) (depending on the exact height of fill at the location of the stress gages) is distributed between the relatively stiff columns and the weaker soils in general accordance with their relative stiffness. About 40 kPa (840 psf) was found transmitted to the soil, and about 400 kPa (8.4 ksf) transmitted to the columns. For the approximately 40% coverage produced by the panels, theory would suggest that the columns would accumulate about twice the average stress applied by the fill. That is, the measured column stress, as predicted by theory, should be between about 400 kPa (8.4 ksf) and about 500 kPa (10.4 ksf). The stress on the soil between the panels is predicted by theory to be between about 50 – 60 kPa (1040 – 1250 psf). The agreement between the simple theory used for these calculations and the measurements appears excellent.

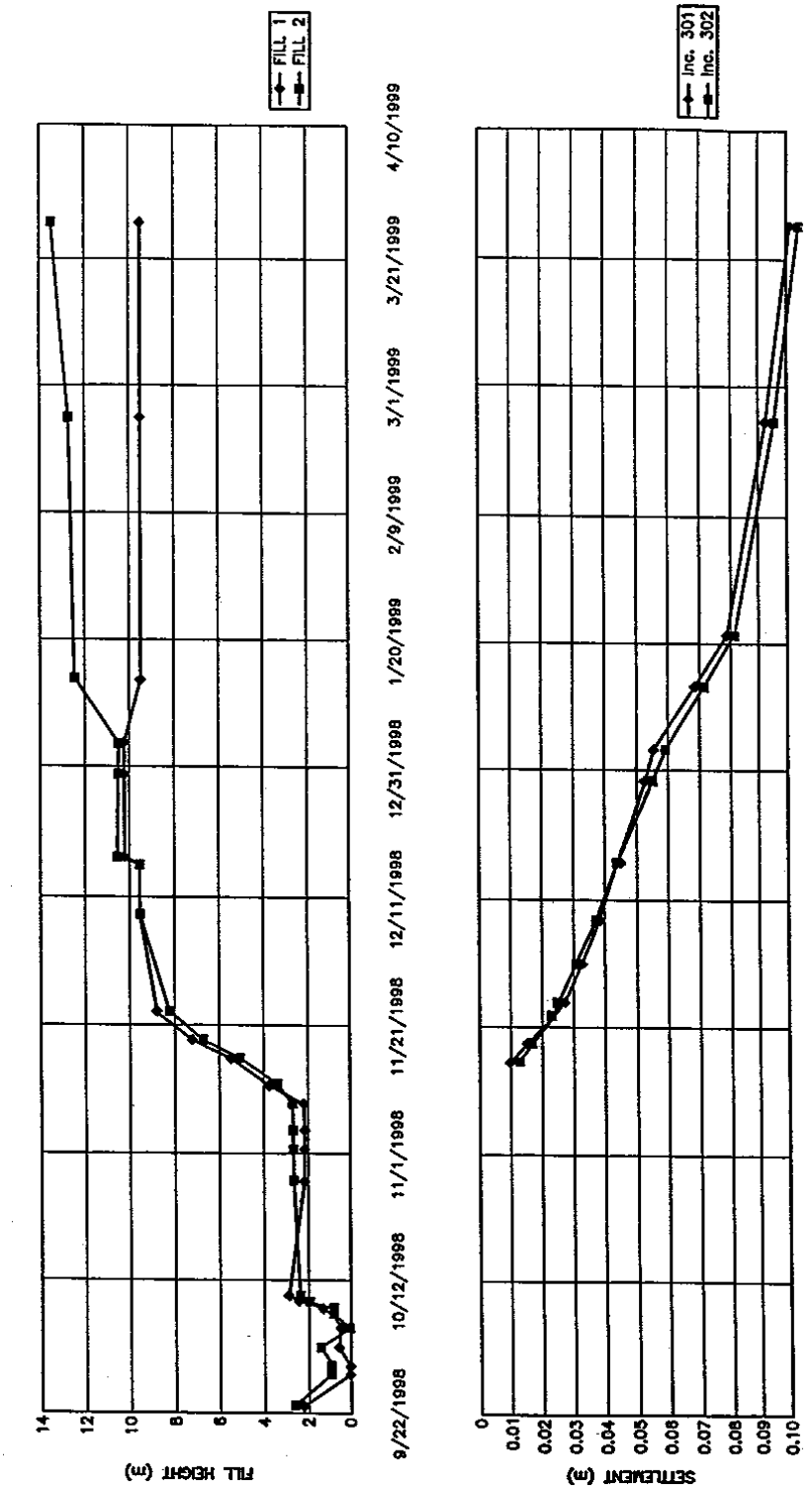

Figure 24. Fill Height and Maximum Settlement vs. Time.

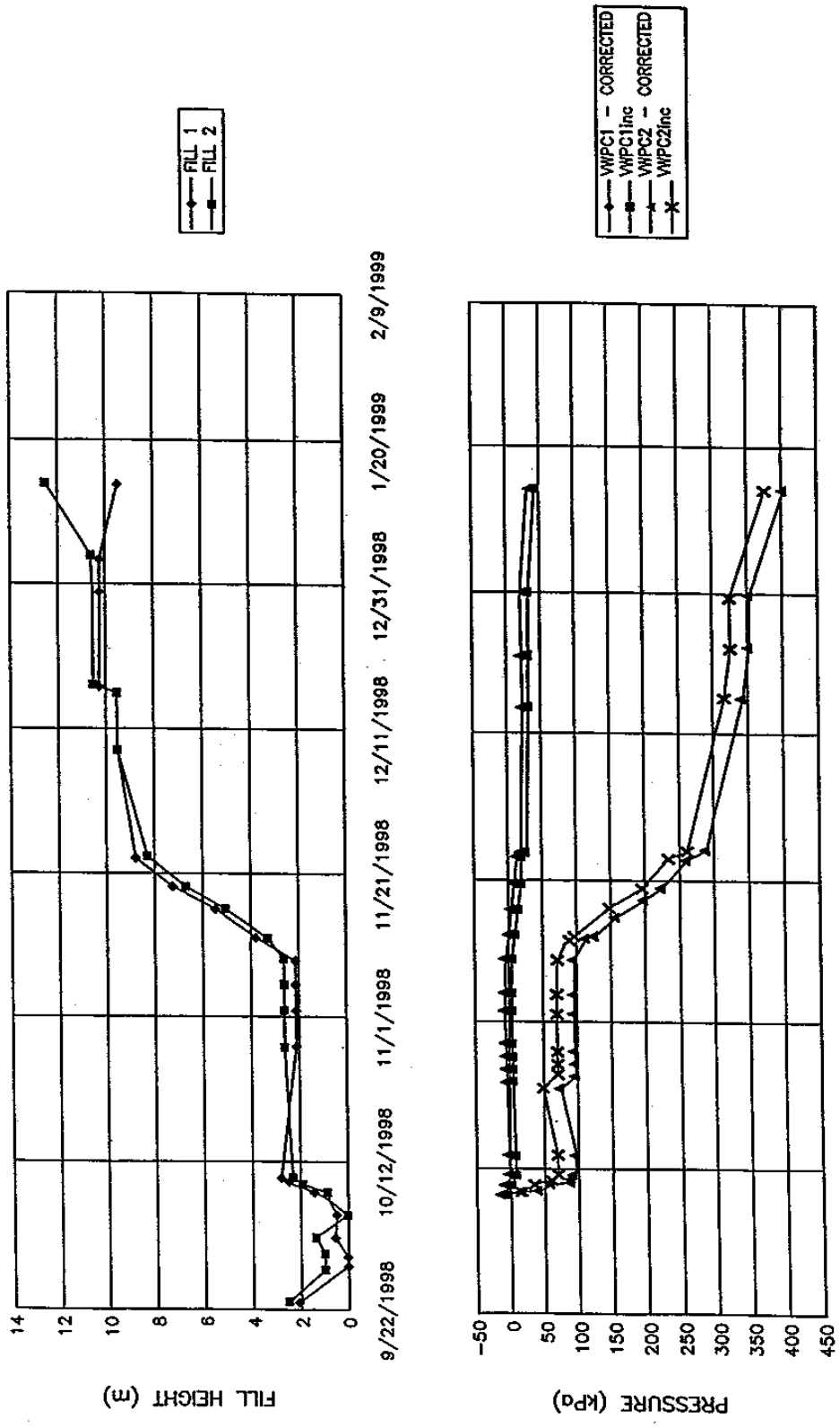

Figure 25. Fill Height and Stress on Load Cell vs. Time.

Conclusions

The performance of Wall SS-01 for I-15 in Salt Lake City, Utah, has confirmed the validity of the design methodology to stabilize weak ground and produce stabilized soils that have an increased but moderate strength and a substantially reduced compressibility. The total settlement under load has been shown to been reduced significantly by the lime-cement column ground stabilization, and the expectations are that the time for the total settlement to occur will also be shown to have been reduced significantly. It is of special interest that the performance measurements made by UDOT Research appear to have confirmed the validity of some of the simple assumptions used in the design of these columns.

The data show that the improvement in ground strength provided by the construction of lime-cement columns for I-15 has permitted the safe construction of an embankment 12 – 15 m (40-50 ft.) high (including surcharge) with single-stage construction. This embankment is significantly higher than could have been constructed on untreated ground. It was constructed without the waiting period that would have been required if wick drains and staged construction had been utilized.

Figure 26 shows the equipment installing the lime-cement columns.

Figure 26. I-15, Lime-Cement Column Construction.

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