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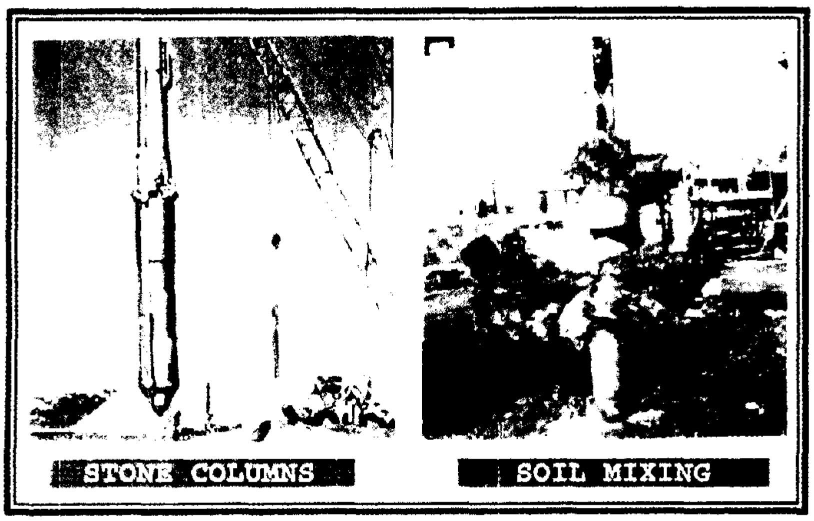
Demonstration Project 116

GROUND IMPROVEMENT TECHNICAL SUMMARIES

Volume II



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16. Abstract

This manual was developed as a reference document for FHWA Demonstration Project No. 116 Ground Improvement Methods. The objective of DP 116 is to mainstream the use of ground improvement technology. The manual is comprised of nine stand alone Technical Summaries on specific ground improvement methods.

The technical summaries reflect current practice in design, construction, contracting methods and quality procedures. The individual technical summaries are intended to serve as a primer on a particular subject; containing an executive summary of technical information which provides concise and specific guidance to users who desire further information. Technical summaries 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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MECHANICALLY STABILIZED EARTH WALLS, REINFORCED SOIL SLOPES, SOIL NAILING, WICK DRAINS, LIGHTWEIGHT FILL, DYNAMIC COMPACTION, DEEP SOIL STABILIZATION, GROUTING, STONE COLUMNS, VIBROCOMPACTION	No restrictions.

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Length	mile yard foot inch	km m m mm	1.609 0.9144 0.3048 25.40
Area	square mile acre acre square yard square foot square inch	km^2 m^2 hectare m^2 m^2 mm^2	2.590 4047 0.404 0.836 0.092 645.2
Volume	acre foot cubic yard cubic foot cubic foot 1000 board feet gallon cubic inch	m^3 m^3 m^3 L (1000 cm^3) m^3 L (1000 cm^3) cm^3	1233 0.764 0.028 28.32 2.36 3.785 16.39
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Force/Unit Area, Pressure, Stress, Modulus of Elasticity.	lbs/in^2 kips/in^2 lbs/ft^2 kips/ft^2	kPa MPa Pa kPa	6.895 6.895 47.88 47.88
Force/Volume Unit Weight	lbs/ft^3 kips/ft^3	N/m^3 kN/m^3	157.1 157.1
Bending Moment, Torque, Moment of force	ft·lb ft·kip	N·m kN·m	1.356 1.356

[6] Underline denotes exact conversion. All others conversion factors on this page are rounded to four significant figures.

Quantity	From English Units	To SI Units	[7]Multiply By
Moment of Mass	lb·ft	kg·m	0.1383
Moment of Inertia	$lb \cdot ft^2$	$kg.m^2$	0.042 14
Second Moment of Area	in^4	mm^4	416 200
Section Modulus	in^3	mm^3	16 390
Work	lb·ft	N·m	1.355 818
Energy	ft·lb	J	1.355 818
Power	ton (refrig) Btu/s hp (electric) Btu/h	kW kW W W	3.517 1.054 745.7 0.2931
Volume Rate of Flow	ft^3 cfm cfm mgd	m^3/s m^3/s L/s m^3/s	0.028 32 0.000 472 0.4719 0.0438
Temperature	°F	°C	(°F-32°)/1.8
Velocity, Speed	ft/s	m/s	$\underline{0.3048}$
Acceleration	ft/s^2	m/s^2	$\underline{0.3048}$
Momentum	lb·ft/sec	kg·m/s	0.1383
Angular Momentum	$lb \cdot ft^2/s$	$kg \cdot m^2/s$	0.042 14
Plane Angle	°	°	no change

[7] Underline denotes exact conversion. All others conversion factors on this page are rounded to four significant figures.

PREFACE

One of the major functions of geotechnical engineering is to design, implement and evaluate ground improvement schemes for infrastructure projects. During the last two decades 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 has developed these Technical Summaries in connection with Demonstration Project No. 116, Ground Improvement Methods. The primary objective of Demonstration Project 116 is to enhance the acceptance and implementation of ground improvement methods by the transportation community. 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 wick drains, soil mixing, stone columns, lightweight fill materials, vibrocompaction, dynamic compaction, soil nailing, mechanically stabilized earth walls, reinforced soil slopes, and micropiles.

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, and form a seepage cutoff and fill voids.

This demonstration project 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. Services provided under the scope of Demonstration Project No. 116 include the following:

- 2 1/2-day workshop on all ground improvements covering selection, preliminary design, construction methods, contracting methods and construction monitoring and inspection.

- Focused 1-day seminars on one or two ground improvement methods.

- Site assessment, design development and review, specification development, and construction trouble-shooting, inspection, and monitoring.

- Experimental evaluations of unusual and major applications.

A second purpose of equal importance is to serve as the FHWA reference for highway projects involving ground improvement.

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 and B. Christopher, FHWA-SA-96-071.

- *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. Barkdale and R. Bachus, FHWA-RD-83-02C.

- *Lightweight Filling Materials,* Permanent International Association of Road Congresses, 1997.

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

The authors further acknowledge the efforts of the Technical Working Group members 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
Dr. John Horvath	-	Manhattan College
Mr. Peter Osborn	-	FHWA
Mr. Richard Sheffield	-	Mississippi DOT
Mr. Michael Simac	-	Ground Improvement Technologies
Mr. Ed Tavera	-	Louisiana DOT

Lastly, the authors wish to thank the clerical and computer graphics staff of Earth Engineering and Sciences, Inc. and Hayward Baker Inc. for their vital contributions and significant effort in preparing this manual.

INTRODUCTION TO TECHNICAL SUMMARIES

Ground improvement technologies are geotechnical construction methods to alter poor ground conditions to meet project requirements, where replacement or bypass of such conditions is not feasible.

Ground improvement has 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 and,
- to increase resistance to liquefaction.

These functions can be accomplished by modifying the soils character with or without the addition of foreign material. The ground improvement methods for which Technical Summaries have been provided can be divided in the following categories:

1. Consolidation
   - by Wick Drains, section 1. Accomplished by accelerating foundation consolidation.

2. Load Reduction
   - by use of Lightweight Fills, section 2. Accomplished by reducing the loads on foundations.

3. Densification
   - by Vibrocompaction, section 3. Accomplished by densification of loose sands by vibratory methods.
   - by Dynamic Compaction, section 4. Accomplished by densification of loose granular/debris by impact methods.

4. Reinforcement
   - by Mechanically Stabilized Earth and Reinforced Soil Slopes, section 5. Accomplished by internally reinforcing fill retaining structures.
   - by Soil Nailing, section 6. Accomplished by internally reinforcing cut retaining structures.
   - by Stone Columns, section 7. Accomplished by reinforcing soft foundations of silts/clays.

5. Chemical Stabilization
   - by Soil Mixing, section 8. Accomplished by physio-chemical alteration of foundation soils to increase their tensile/compressive strength.
   - by Grouting, section 9. Accomplished by densification and/or replacement.

The purpose of these Technical Summaries is to introduce the outlined ground improvement methods and applications primarily to generalists involved in project development. The summaries outline methods, function, applications, benefits, limitations and summarize design issues. Outlined are 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.

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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 and 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 addition, this chapter introduces the new technology of vibro concrete columns, an extension of the bottom-feed vibro technique and Geopiers an intermediate depth foundation 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 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.

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 is to identify problems that have been successfully solved by the stone column method and to synthesize the current state-of-the-art 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 can solve a specific problem. It is the intent of this document to serve as a reference on stone columns and how they may solve a specific problem by discussing the construction, utilization, and limitations of vibro-replacement and vibro-displacement stone columns, vibro-concrete columns and Geopiers.

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 over 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 compactibility of a soil depends mainly on its grain size distribution. 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 compactible by vibro-compaction. It is for these type 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 over 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 into 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 are being built on sites with poor soil.

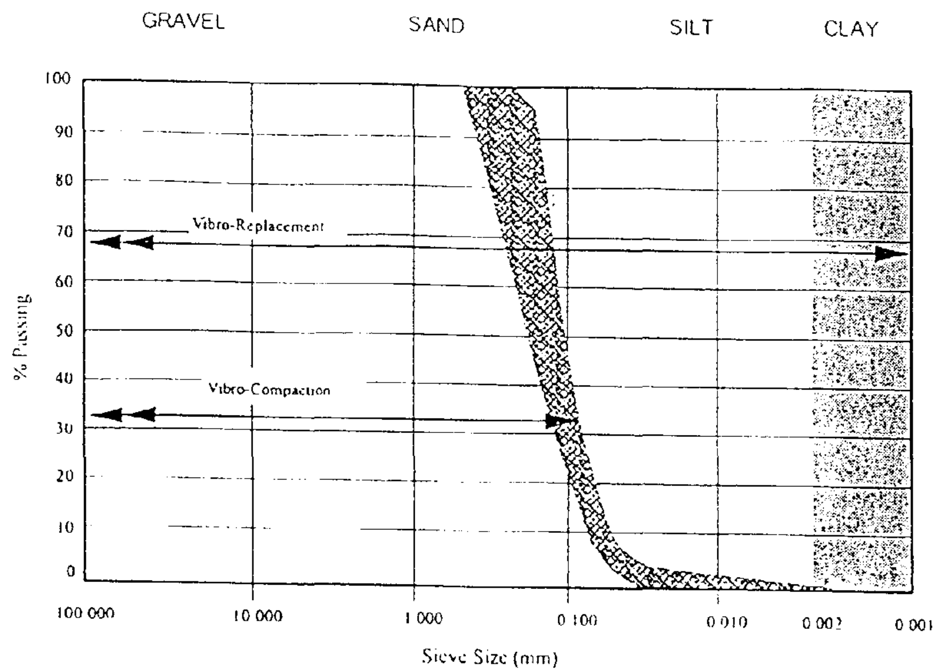

Figure 1. Grain size distribution curves.

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 now account for a significant proportion of stone column construction, the wet method is used extensively. Vibro-concrete column and Geopiers can be considered to be emerging technologies when applicable.

Where environmental concerns are strong, the dry process will typically be required. However, the wet process is more economical, if these concerns are not relevant to the project. Vibro concrete column construction is a dry-construction, displacement technique, illustrated in figure 4.

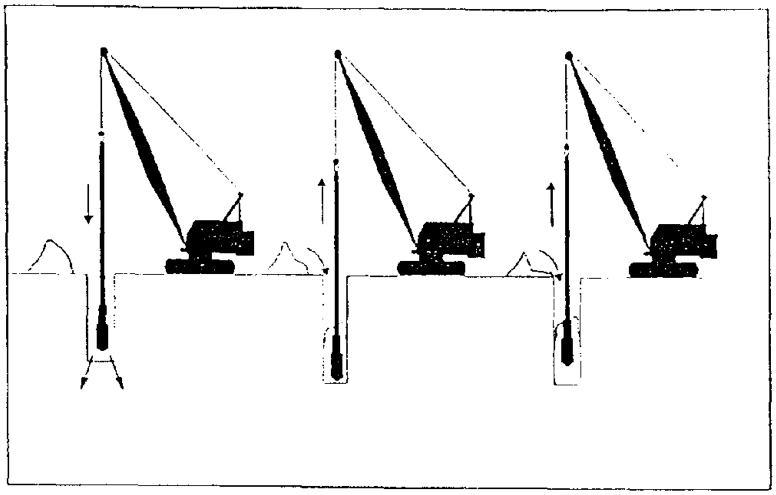

Figure 2. Top feed vibro-replacement.

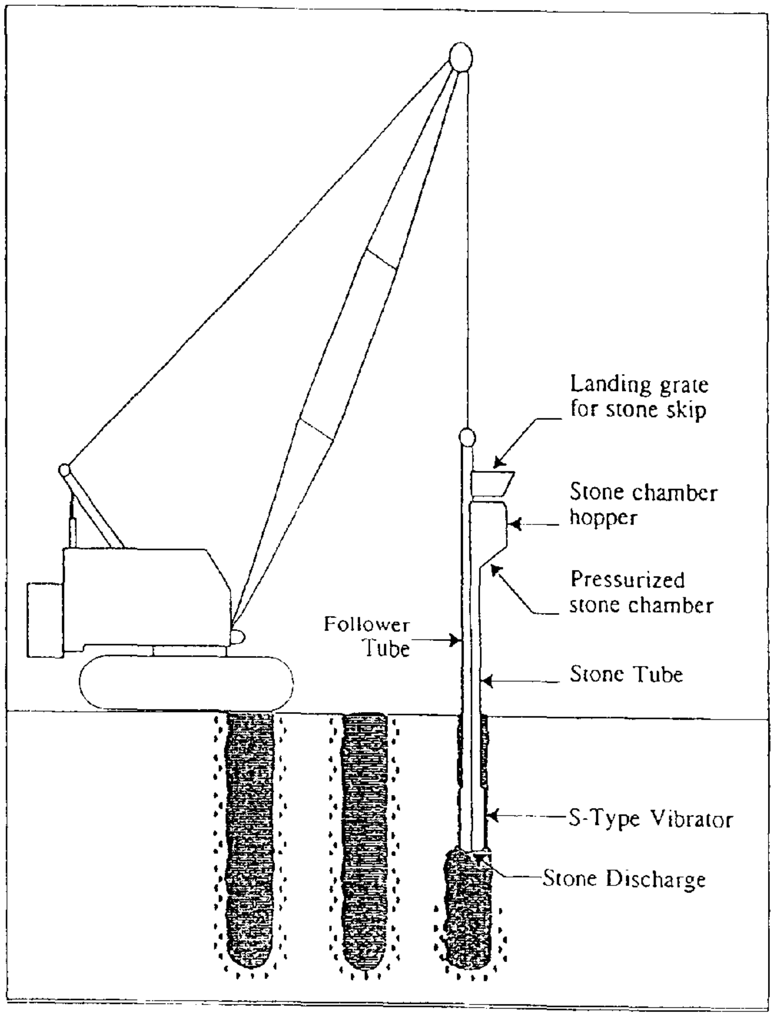

Figure 3. Bottom feed vibro displacement.

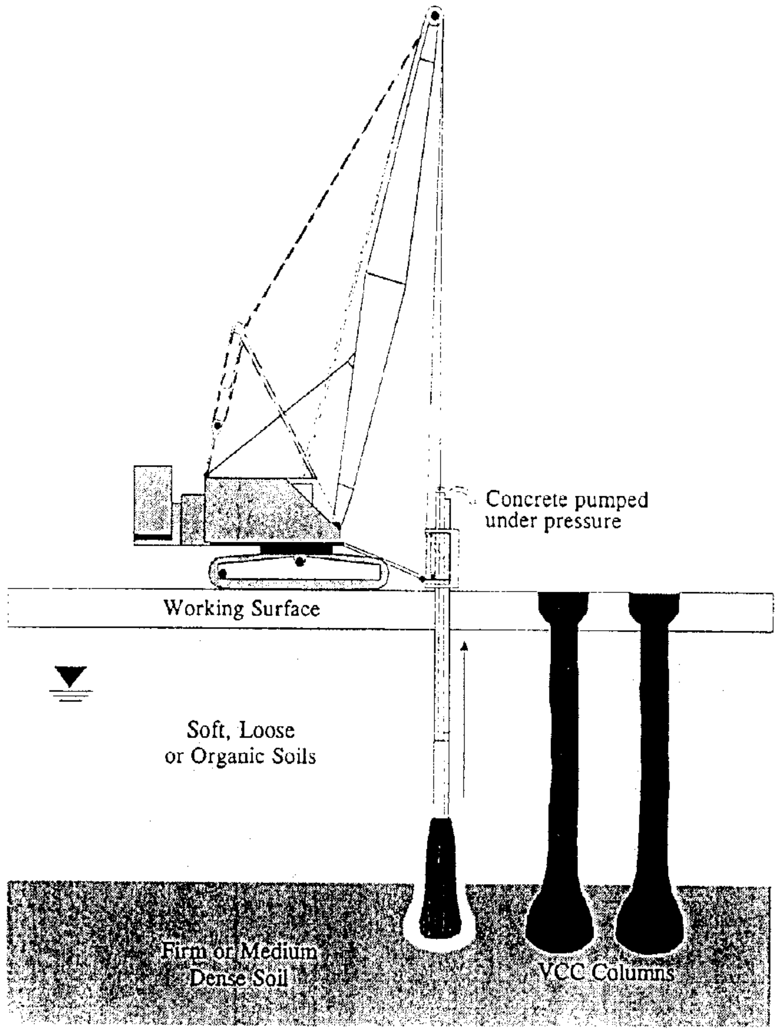

Figure 4. Vibro-concrete columns.

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 both the stone column and the soil. This method is best suited for sites with soft to firm soils with undrained shear strengths of 10 to 50 kN/m^2 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 auxiliary 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. 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]

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.

d. Geopiers™

The concept of short dug pits backfilled with aggregate to support structures is not new and has been previously documented in the literature.[1] Some refinements and improvement to this basic technique have been introduced in the last decade under the trade name Geopier™. The Geopier elements are constructed by drilling or excavating a hole to create a cavity, removing a volume of compressible subsoil, then building a bottom bulb of clean open graded stone while prestressing the soils beneath the bottom bulb. The shaft is built on top of the bottom bulb, using well-graded or clean aggregate placed in 300 mm lifts. Densification throughout is accomplished by the impact ramming action of a modified hydraulic hammer with a beveled taper head impacting energy from 34,000 to 90,000 kg-m per 300 mm lift within a 760 mm diameter pier.[13,15]

This process of making the aggregate pier is subject to a United States patent issued in 1993.[14] The most economical application for this type of construction appears to be in connection with support of structures or retaining walls on relatively shallow compressible foundations.

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 1983 "FHWA Stone Column Manual" suggested the following four potential highway uses:

a. Embankment stabilization.
b. Bridge approach fills stabilization.
c. Bridge abutment and other foundation support.
d. Liquefaction mitigation.

These applications are still the basis for current usage. The data base of successful projects continues to expand, and, with the development of dry stone column construction techniques, environmental considerations can be addressed.

a. Embankments

Typical applications 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 for 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 wet land 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 of stone columns to support mechanically stabilized earth for the U.S. 77 overpass situated in Brownsville, TX and in 1990, the Texas D.O.T. utilized 12,800 m of 4-to 6-m-deep stone columns for Brownsville road over U.S. 77.[9]

Under favorable conditions, stone-column-supported embankments can be constructed to greater heights than a conventional approach embankment 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

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 working close to structures. As 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 organic material such as peat are present, vibro-concrete columns can also be a technically feasible and economical solution.

Geopiers provide an alternate construction technique with engineering characteristics similar or superior to stone columns, to 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 of less than 3.5 m using small equipment and tampers or up to 6 m using larger equipment and tampers.

Disadvantages

Stone columns are not a solution for all soft soil problems. Strata of peat, mulch, and other organic materials, 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 predrilling prior to installation of stone columns.

A major disadvantage is often cost, when compared to other solutions. 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.

2.3 FEASIBILITY EVALUATIONS

The stone column technique of ground treatment has proven 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, plasticity of the soil, pre-densification relative densities, vibrator type, stone shape and durability, stone column area, and spacing between stone columns. Experience has shown that soils with less than 15 percent passing a 74μ (#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, or pier.

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 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 of the surface and ranging in shear strength from 15 to 50 kN/m^2.

3. Special care must be taken when using stone columns in highly sensitive soils and in soils containing organics and peat lenses or layers with undrained shear strength of less than 10 kN/m^2. 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 to 50 percent of the unimproved ground response.

5. Stone columns have been used in clays having minimum (not average) undrained shear strengths as low as 7 kN/m^2. 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 to 100 kN/m^2. 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 loads 20 to 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. Improvement of Settlement Characteristics

Stone columns act similarly to prefabricated, vertical drains (wicks) in decreasing the distance which water has to flow in the radial direction for primary consolidation to occur. As a result, installation of stone columns 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 will greatly accelerate the gain in shear strength of cohesive soils as primary consolidation occurs. In addition, total settlement is reduced as the stone 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 stone column 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 could be evaluated using field pumping tests. Piezometer or well point permeability tests are alternatives that should yield horizontal permeabilities equal to or less than those obtained from pumping 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 to support embankment loads. Secondary settlement will therefore be an important consideration in many stone column projects. Because of the relatively short time usually required for primary consolidation to take place in stone column reinforced soils, secondary settlement is even more important than if columns are not used. Neither stone columns nor vertical drains effect 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 will be 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 concept of soil liquefaction has been understood 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) Increase the soil density.
2) Increase the confining pressure.
3) Increase the material stiffness by installation of stone columns.
4) Control and/or prevention of pore pressure development.

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)

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 an increase in soil density only; the ability of the stone column to act as a drain and the stiffness of the stone column are not usually accounted for in the design approach. 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 column 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 Geopiers and Vibro-Concrete column, the following alternative methods 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 12 percent with clay less than 3 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. Vibro-concrete columns may be an effective solution when constructed in very low strength soil types and Geopiers considered when shallower improvement is feasible.

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 to 60 mm in size) is used. This size backfill passes easily around the vibrating probe while still in the hole. The larger size 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 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 in size to avoid blockage of the equipment.

For geopier construction, No. 57 stone or other graded aggregate types are commonly specified.

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 5).

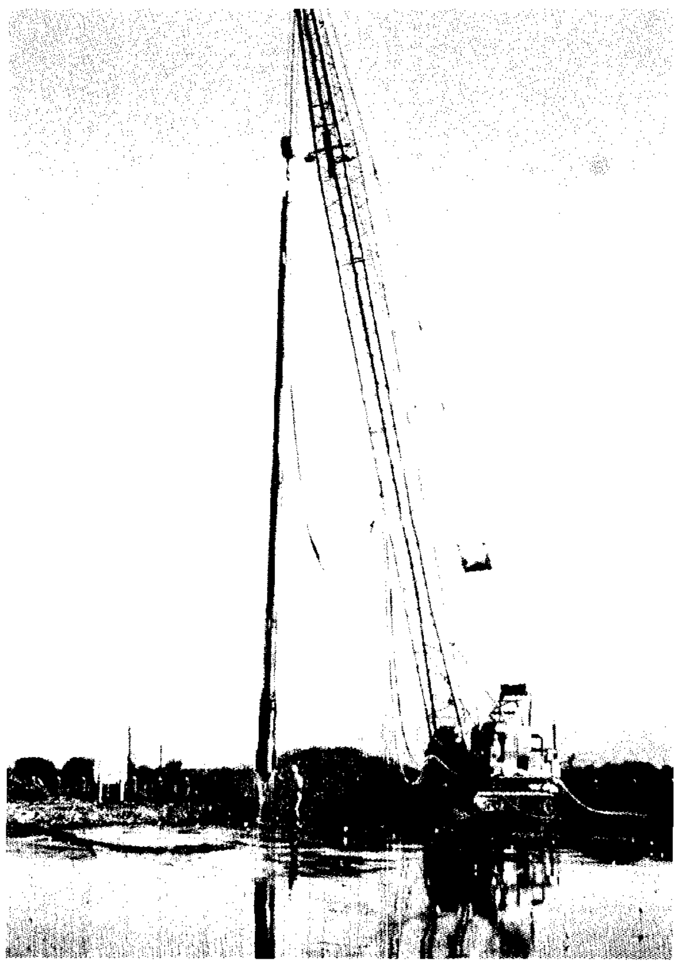

Figure 5. Elastically suspended vibrator.

- 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 to 460 mm and in length from approximately 3.0 to 4.9 m. A cross section of a typical vibrator is shown in figure 6.

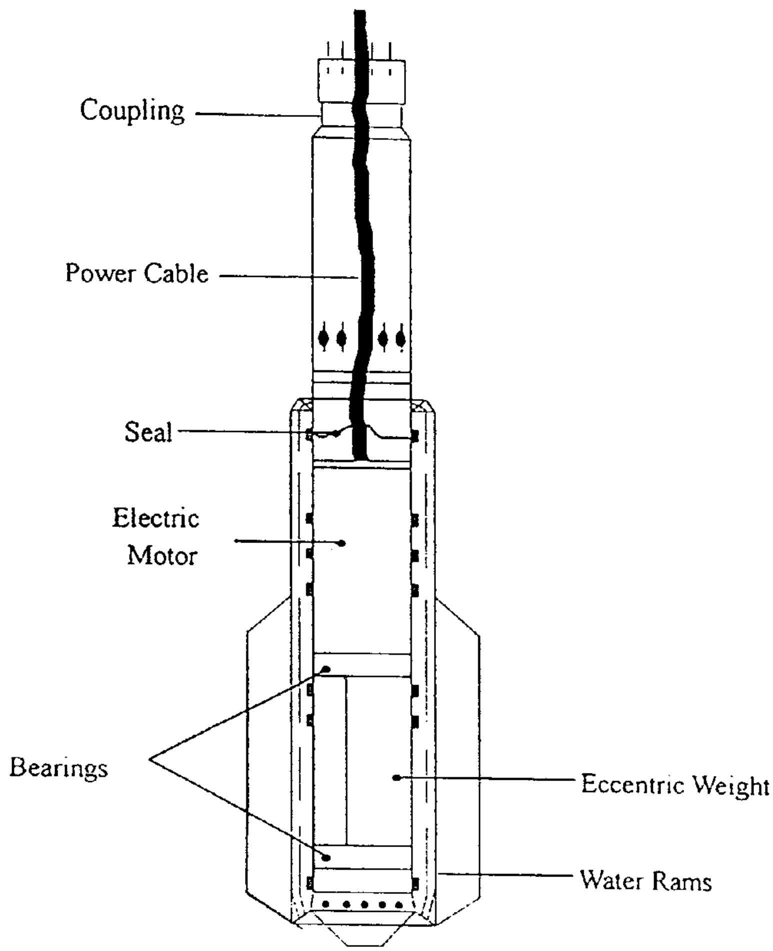

Figure 6. Vibrator cross section.

Horizontal vibrations are produced close to the base of the vibrator and are induced by rotating eccentric weights mounted on a shaft and driven by a motor located in the upper part of the vibrator casing. Both electric and hydraulic power can be used to power the motor. Early units were driven by motors in the 22-to 60-kW range, but more recent machines develop up to 125 kW. Centrifugal forces of up to 30 tons at frequencies varying from 1200 to 3000 rpm are currently achieved. Abrasion resistant wear plates are added to the sides of the vibrator, protecting it from excessive wear during raising and lowering from the ground. Fins located on the sides of the vibrator reduce rotation. Follower or extension tubes, typically of a similar or smaller diameter to the vibrator unit, are attached to it and allow treatment of soils at depth. An elastic coupling is used to isolate the vibrator from the extension tubes and to prevent vibration travelling 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-probe can be easily handled using a 36,000 kg crane with a 12-m 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 mounted on the leads. 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 thru a 150 mm hose instead of a skip along a leader.

Recent development includes an instrumentation package that provides a continuous record of construction data for each stone column. Measurements of depth, power consumption, and stone consumption are recorded against time and provided on a print out 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.

Typical equipment to insall stone columns is illustrated in figures 7-11.

Geopier impact ramming equipment is shown on figure 12.

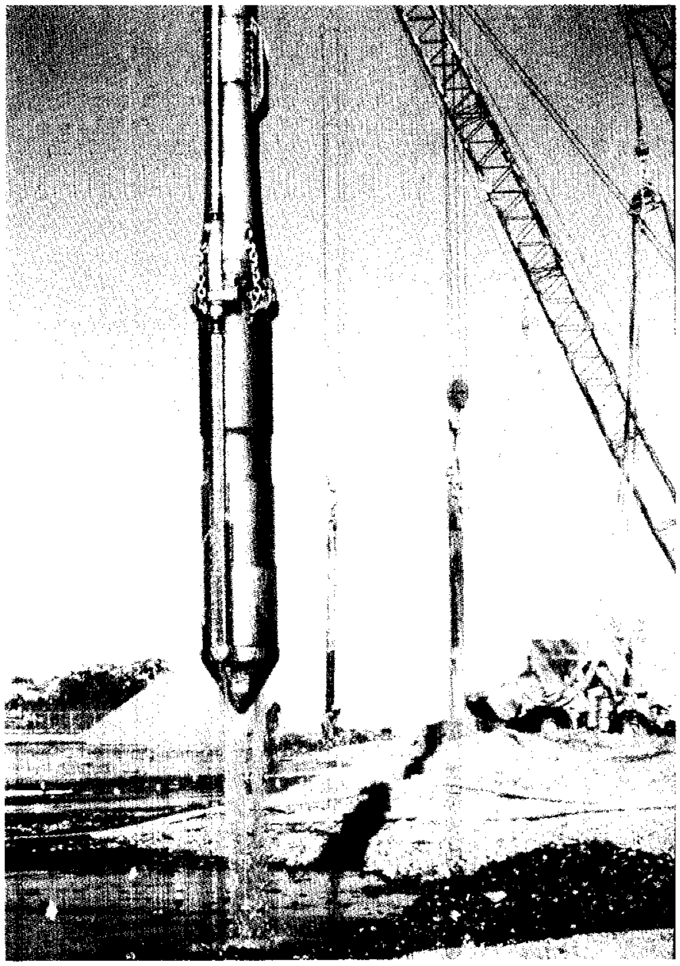

Figure 7. Vibrator in use for wet, top feed.

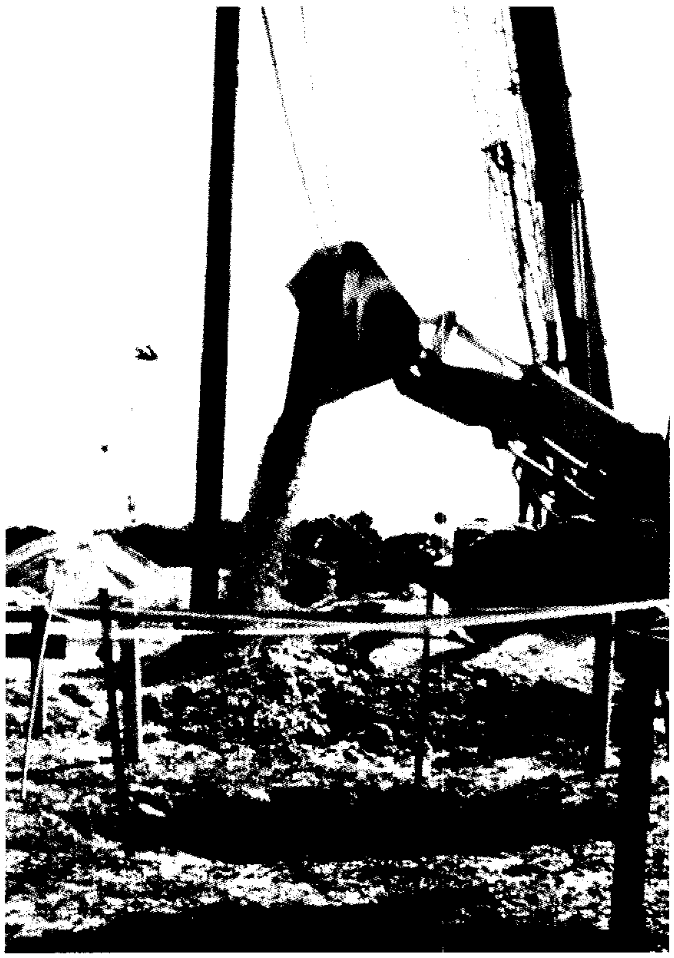

Figure 8. Conventional front-end loader placing stone for top feed vibro-replacement.

Figure 9. Truck mounted crane utilized for top feed vibro-replacement.

Figure 10. Bottom feed vibro rig with stone hopper.

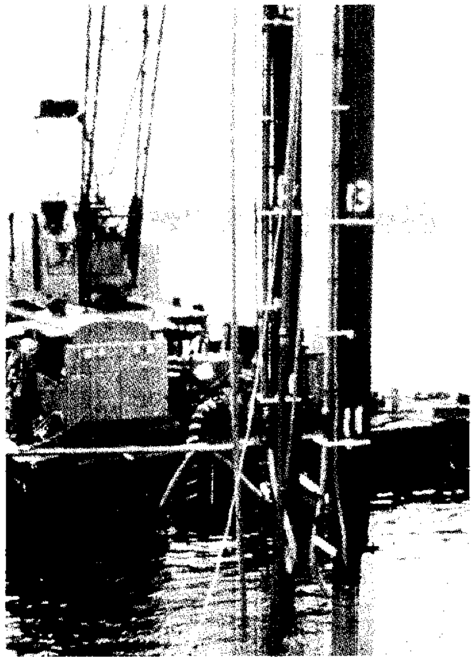

Figure 11. Barge mounted stone column equipment.

Figure 12. Geopier impact ramming equipment.

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 Reinforcement/Treatment: Developments (1987-1997), discusses design concepts to reduce liquefaction potential.[3]

In practice, the design of stone columns is to a large extent semi-empirical. Specific state-of-the-art 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 judgement.

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 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 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 figure 13 and 14. 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]

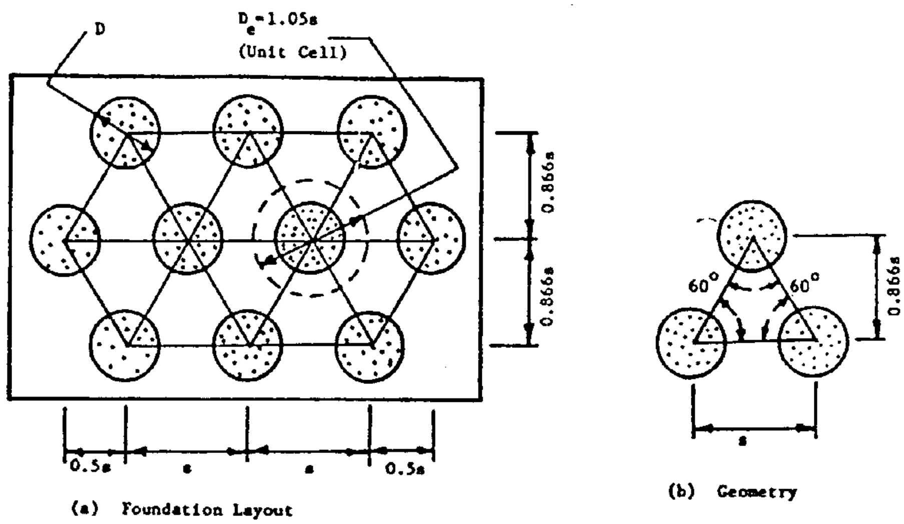

Figure 13. Equilateral triangular pattern of stone columns.

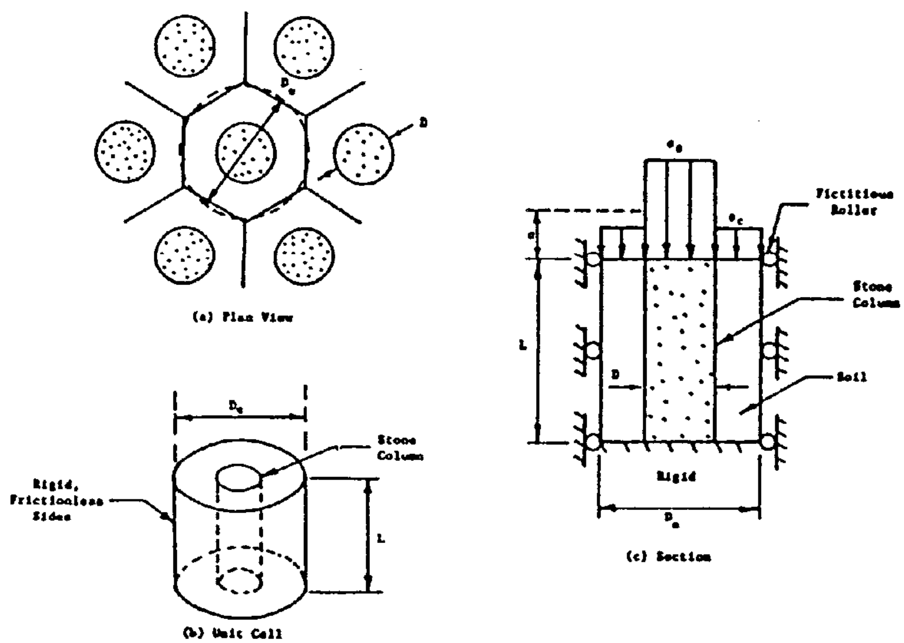

Figure 14. Unit cell idealization.

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*, a_s, is defined as the fraction of soil tributary to the stone column replaced by the stone:

$$a_s = \frac{A_s}{A} \tag{1}$$

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 m and 1.2 m, but are typically in the range of 0.9 to 1.1 m 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 to 3.5 m. For footing support they are installed in rows or clusters. For both footing or wide area support they should 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 spacing very tight. For stronger in-situ soils and large 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 assummed or determined, the stress on the stone column, σ_s, and on the surrounding soil, σ_c, can be calculated for each replacement ratio, a_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

$$q = \sigma_s(a_s) + \sigma_c(1 - a_s) \tag{3}$$

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 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 15, 16, and 17. Caution should be given to avoiding local bulging failures due to very weak, or organic layers of limited thickness (figure 16). 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.

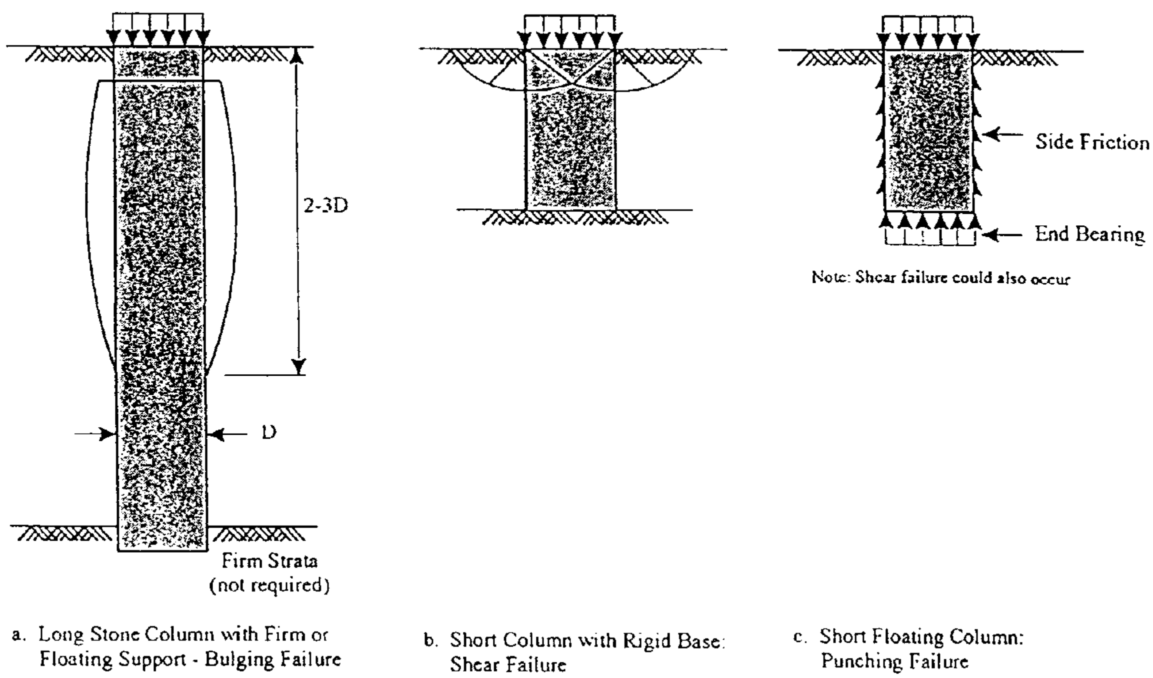

Figure 15. Failure modes of a single stone column in a homogenous soft layer.

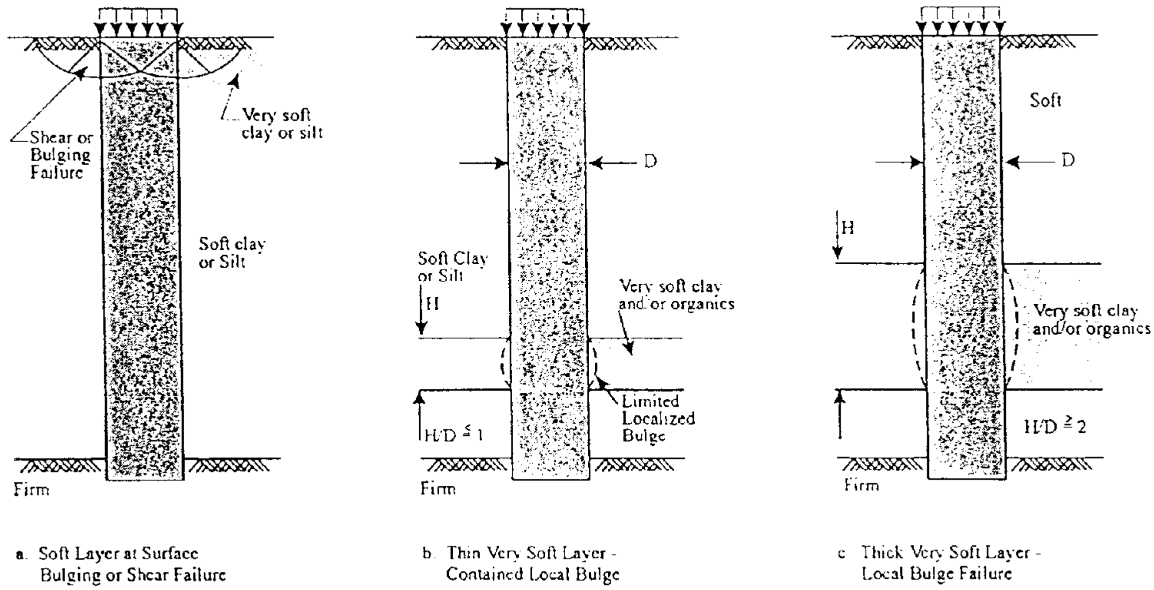

Figure 16. Failure modes of a single stone column in a nonhomogeneous cohesive soil.

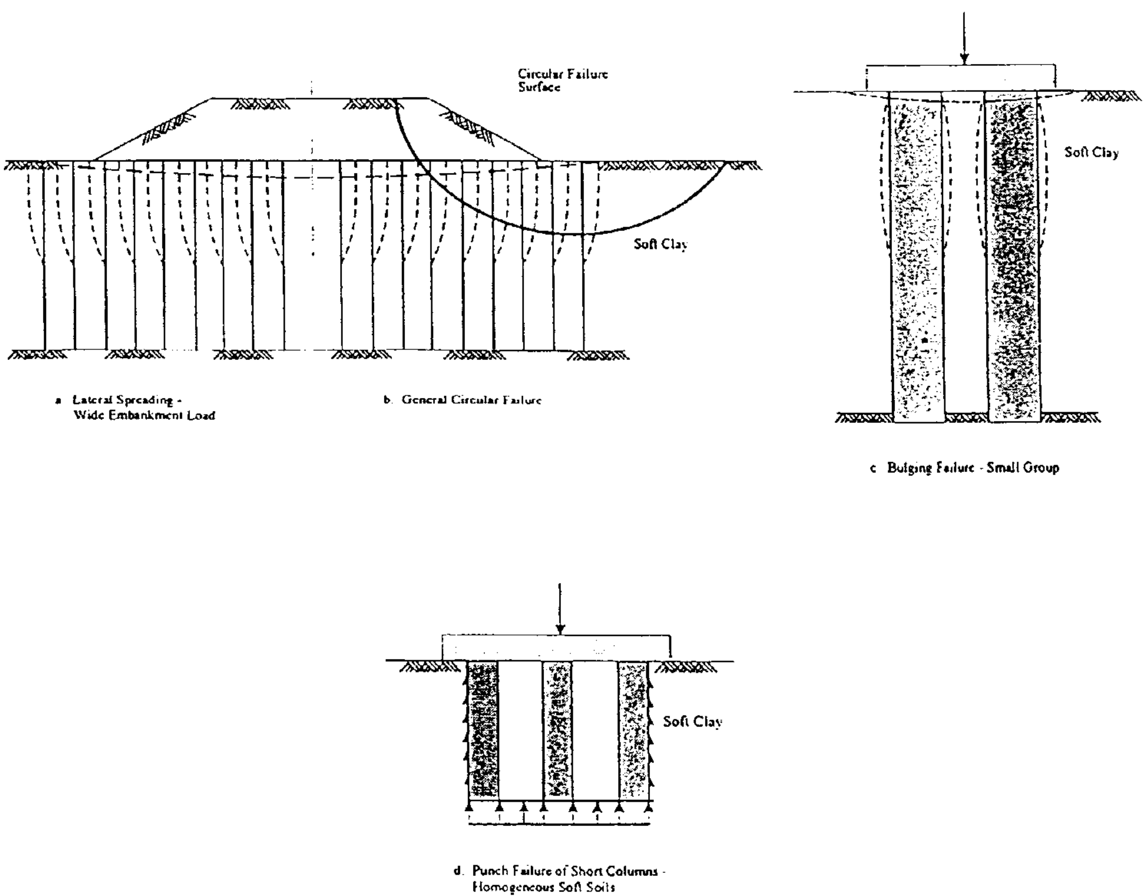

Figure 17. Failure modes of stone column groups.

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 judgement 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.

Typical single column design loads of 20 to 30 tons can be used in soft to medium stiff clays.

For Geopier, 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 elastoplastic 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 18.[24]

It should be noted in figure 18 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 (E_s) or a measure of the strength of the stone column (ϕ) or 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 18 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 sand drains. The effect of disturbance or smear during installation which reduces radial flow can be roughly accounted by reducing the diameter of the column by 50 to 80 percent of its design diameter. A larger disturbance or smear zone should be anticipated with the dry-displacement construction method and for all installations in sensitive clays.

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 19.[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 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]

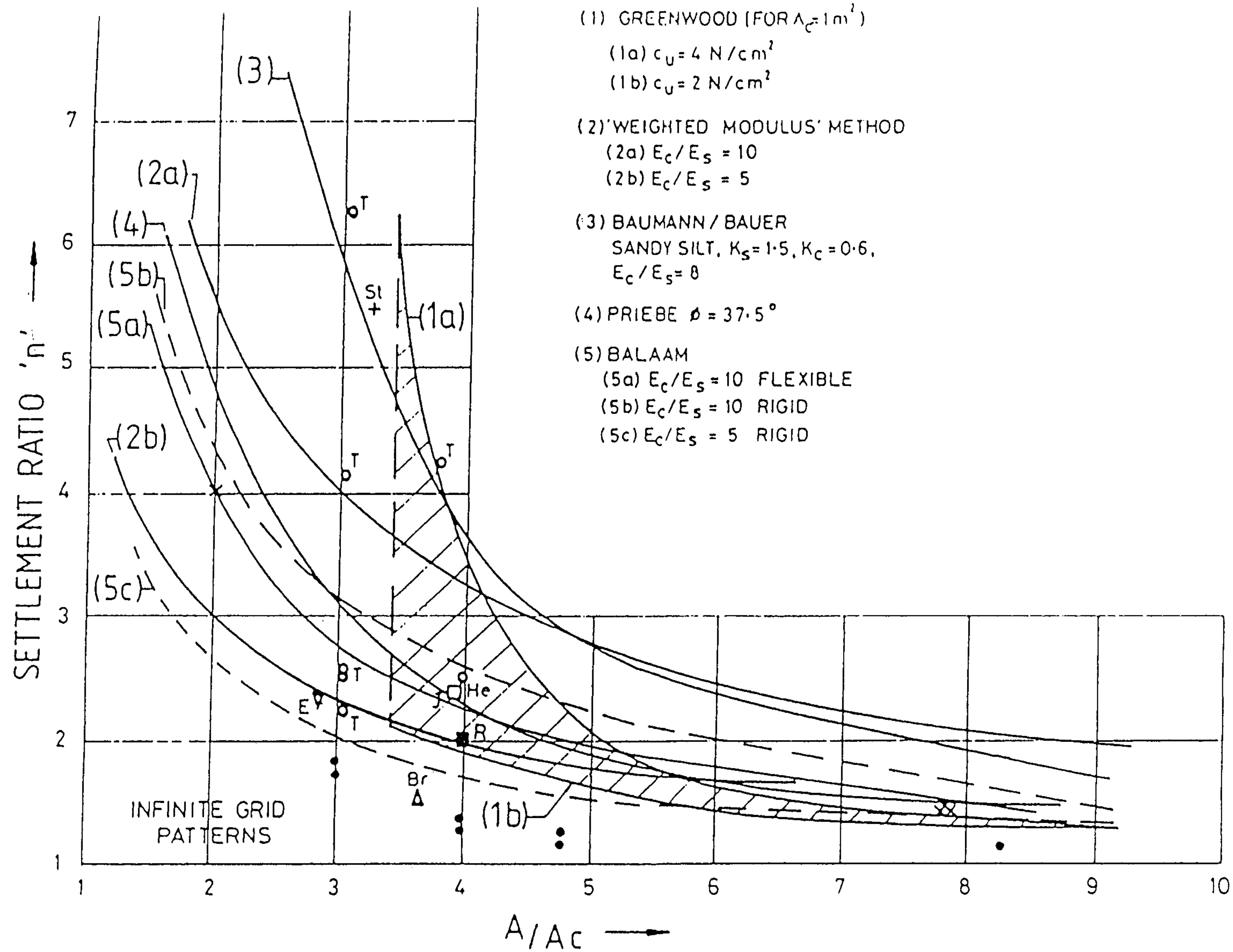

Figure 18. Comparison of elastic theories and field observations.[22]

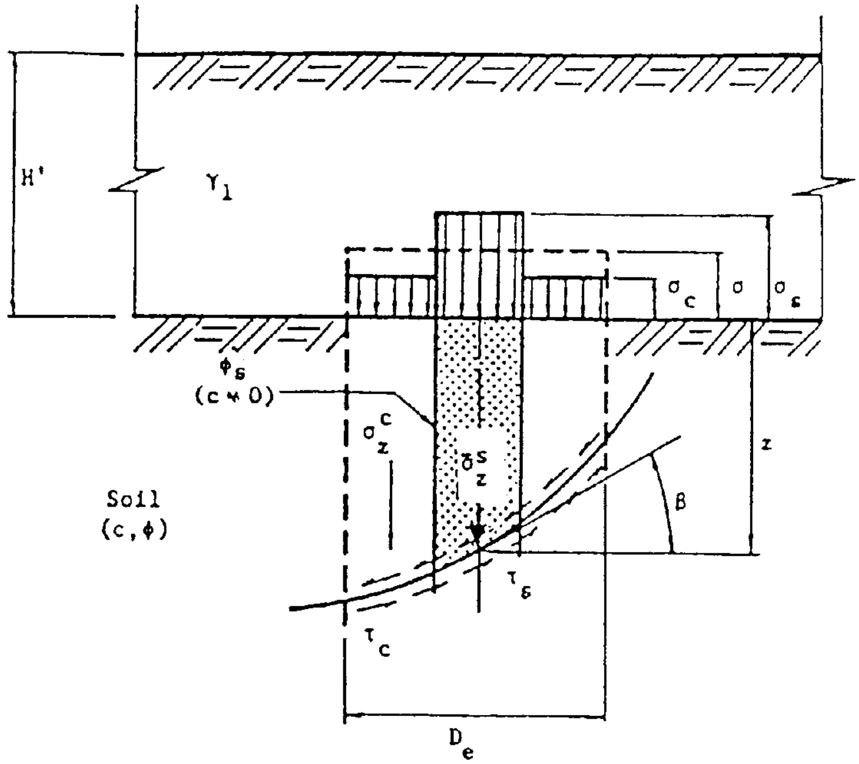

Figure 19. Notation used in average stress method stability analysis.

The average strength and average unit weight parameters (τ; $\bar{\gamma}$) can be determined as follows:[1]

$$\tau = (1 - a_s)\, c_u + a_s\, \sigma_v \tan \phi_s \qquad (7)$$

$$\bar{\gamma} = \gamma_c\, a_s + \gamma_s\, (1 - a_s) \qquad (8)$$

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 to 45 degrees. The lower angles should be considered for gravel mixtures and the higher angles for angular crushed stone mixtures.

Note that in landslide remediation projects the stress ratio is 1 and consequently the strength parameters are essentially a weighted average.

Stability analyses may be performed using a total stress approach by assigning $\phi_c = 0$ 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 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] provide recommendations on how to quantify the benefits of ground improvement and evaluate the actual safety factor against seismic liquefaction. The recommended approach typically is to increase density, provide for pore water dissipation and ensure for the continuing function of the stone column. This mandates that the stone columns be installed using a vibroprobe.

Soil Density

It is well understood that under cyclic loading, pore pressure generation in a dense soil occurs more slowly than in a loose sand, and 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.

The level of densification to preclude liquefaction can be characterized by in situ shear wave in excess of 200 m/s regardless of the cyclic stress ratio (CSR). For CSR of 0.1 shear wave velocities of 150 m/s may be adequate to preclude liquefaction.

Shear wave velocities can be measured in situ by several seismic tests including crosshole, downhole, seismic cone penetrometer, suspension logger etc. Reference 29 provides a complete update and review of current knowledge on liquefaction resistance.

Spacing

Design procedures for the required spacing have not been conclusively demonstrated to date. Mathematical procedures have been suggested by Priebe in Europe and Baez in the United States.(11,12) Priebe suggests that the soil cyclic stress ratio for the site can be reduced by multiplying it by the inverse of the stress ratio (typically 0.25 to 0.5) which is a function of the area replacement ratio. Therefore, spacing can be varied until a cyclic stress ratio not likely to cause liquefaction is calculated.(11)

Baez suggests a mathematical relationship between the ratio of the shear modulus of the stone column and of the soil and the area replacement ratio. He then develops a correlation between the pre-improvement factor of safety against liquefaction and the required area replacement ratio.(12) Based on either consideration a preliminary spacing in the range of 2 to 3 m is generally obtained.

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 this criteria will ensure maximum permeability and prevent piping of the soil:

$$20D_{S15} < D_{G15} < 9D_{S85} \qquad (8)$$

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.

4.4 VIBRO-CONCRETE COLUMNS (VCC)

The VCC technique combines the ground improvement advantages of the vibro systems with the load carrying advantages of piles.

The design of the VCC improved site is essentially the same as would be performed for a driven or bored pile foundation, except that the 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. 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 (< 6m) can be placed into the completed columns.

4.5 GEOPIERS

The design concept used for Geopiers 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 geopiers 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 geopiers 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 geopier 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 to 25 percent of ultimate should be used. A final acceptance criteria of 2.5 cm of settlement at 150 to 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 to 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 test can be performed for quality control purposes. These tests are referred to as proof tests and are used to verify quality control during production. The load applied in the proof test is usually 150 to 200 percent of the allowable/design load.

In-situ testing to evaluate the affect 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 geopier 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.

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 regrade 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 5000 tons, or as required by the engineer, of stone or sand furnished from each source.

Water

Fresh, brackish, sea water, 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 to 480 mm 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 to 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 15 tons gyrating about a vertical axis. The minimum double amplitude (peak to peak measurement) of the probe tip should be not less than 14 mm 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 collapse 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 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 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.

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 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 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.

(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.

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 2.5m 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 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 travel and graduated in 0.025-mm divisions. The tips of the stems of the dial gages should rest on metal plate 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 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 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 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 scales so as to approximately fill a standard size 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 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 mm to 75 mm.

Equipment and Procedure

Specific equipment and installation procedures to achieve the specified criteria are the contractors 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 to 450 mm 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 the 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 to 270 kN per column in soft to medium stiff clays, 2) an average bearing pressure, typically 150 to 200 kN/m^2, 3) a total settlement that is less than a specified value, typically 25 to 100 mm, 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); 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 contractors 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 consist 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 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 120 kW and a minimum centrifugal force of 180 kN of force gyrating about a vertical axis. The minimum double amplitude (peak to peak measurement) of the probe tip shall be not less than 14 mm in a horizontal direction when the probe is in a free suspended position. Limit the air pressure used to a range between 200 to 400 kPA 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 333-mm 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 collapse of the hole and to form a clean stone column.

   d. After penetration to the treatment depth, slowly retrieve the vibrator in 300 to 500 mm 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 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 to 150 mm 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 the 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) 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 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 to 3 m 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. Also the probe should not be lifted completely out of the hole during stroking.

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 to the wet method of construction only 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

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 3m 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 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 to 3 m) 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)

9. Observe that the vibrator passes through the recently placed lift of stone during the first penetration; additional repenetrations 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 to obtain 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 20). 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.

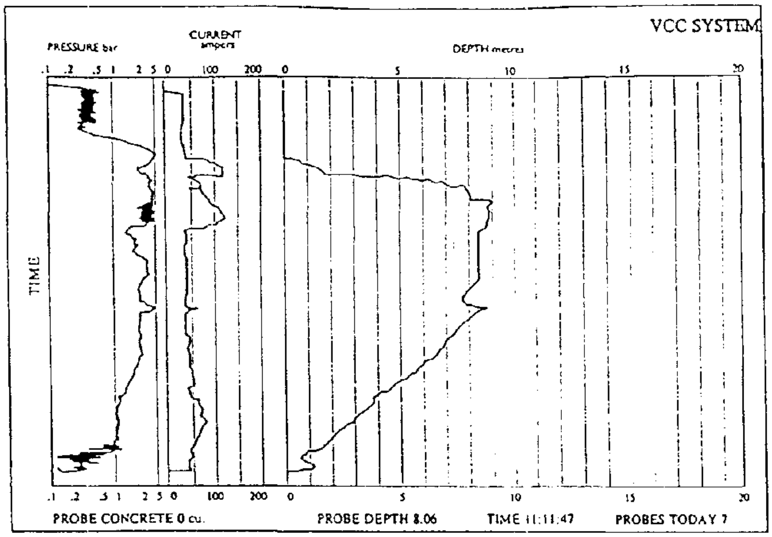

Figure 20. Typical printout providing record of vibro-concrete column construction.

Geopiers

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 geopier construction. A short check list 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 Geopier elements as appropriate
- implementing corrective measures when necessary, with approval of the Geopier foundation designer
- reporting construction activities to the engineer.

Unique to Geopier field testing is a Bottom Stabilization Test (BSTA) which is a method of verifying that the Geopier element being installed has achieved a general stabilization prior to the completion of the installation. It is also a method to determine that production Geopier elements are comparable in quality to load test Geopier elements. 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 Geopier element 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 Geopier element, stop construction, verify the aggregate compaction as described below, redrill the Geopier cavity and rebuild the Geopier, unless directed otherwise by the Geopier 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 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 this test. As a result, these tests tend to be performed on the 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 that the economic feasibility of stone columns may be determined. 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 which 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) Calculating the number of stone columns required by dividing the square meter spacing determined 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 columns 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.

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 ten 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 $600.00 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. Geopiers

At present, Geopiers are typically used for commercial structure foundation support. Limited information indicates that they are typically bid at $400 to $600 per pier or at $3.00 to $6.00 per kN of column load.(17)

Based on the low end lump sum cost per pier and a typical 760 mm diameter pier at a maximum depth of 3.5 m, a minimum linear meter cost of $115 is indicated. On a volume in place basis a cost of $140 per m^3 can be computed. These costs include mobilization and gravel supply.

CHAPTER 7

CASE HISTORIES

Representative case histories of transportation-related construction projects are presented which illustrate the application of stone column technology.

7.1 SLOPE STABILITY IMPROVEMENT

NYDOT Route 22, Wadhams, NY[8]

Along a 67-m 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 downslope. Of particular importance in addressing slope stability improvement alternatives was that the site lay within the Adirondack State Park and within 15 m 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 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.

A minimum 0.3-m-thick 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 21 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 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 per day. Over the course of the stone column installation, total additional movement was 3.3 mm. As of May, 1995, some eight years after project completion, New York D.O.T. reported little to no movement recorded.

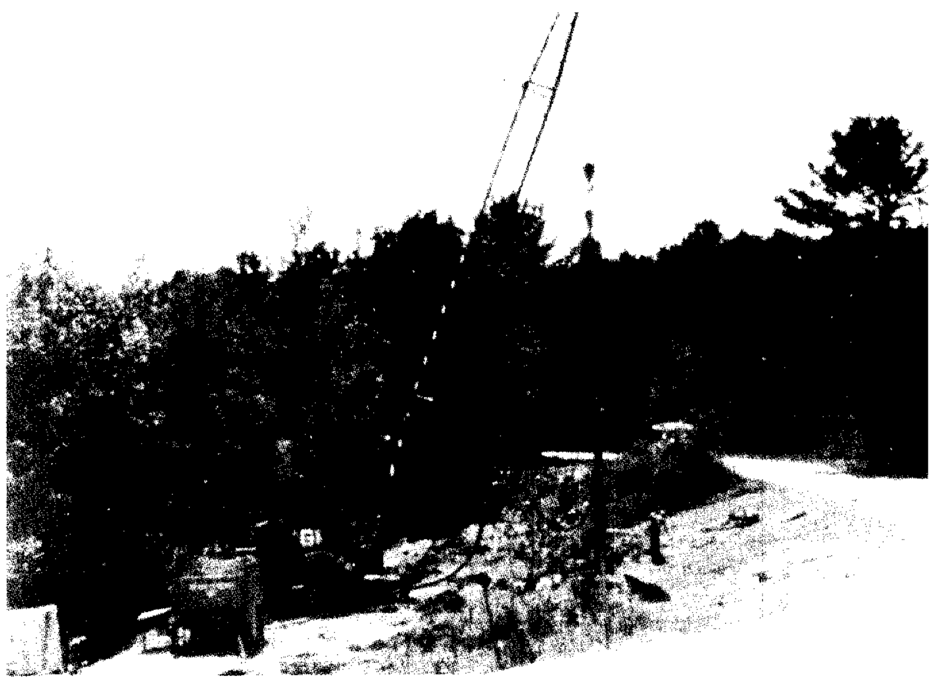

Figure 21. Construction of stone columns, Rt. 22 N.Y.

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 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, and many of the 405-mm square prestressed concrete footings bearing on fill settled 225 to 350 mm. 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 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 spaced 2.4 m apart. The ground improvement scheme developed for the wharf is shown in figure 22 and a project photo on figure 23.

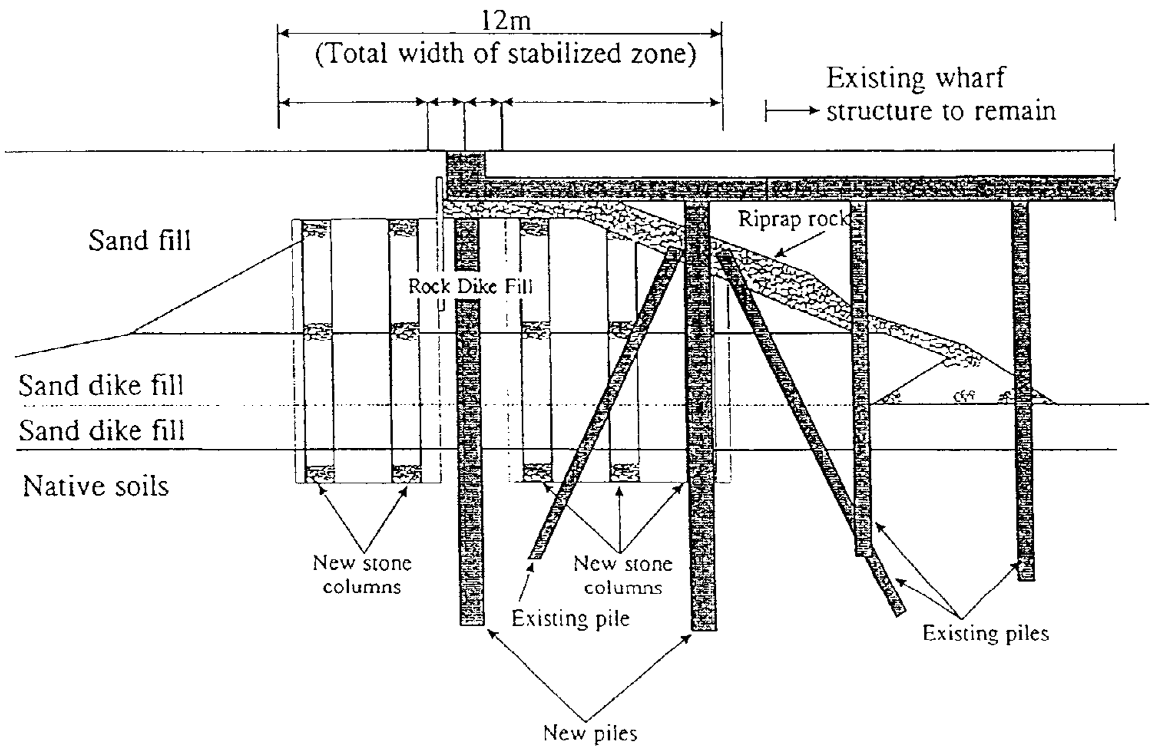

Figure 22. Stone column and new piling for Port of Oakland wharf repair.

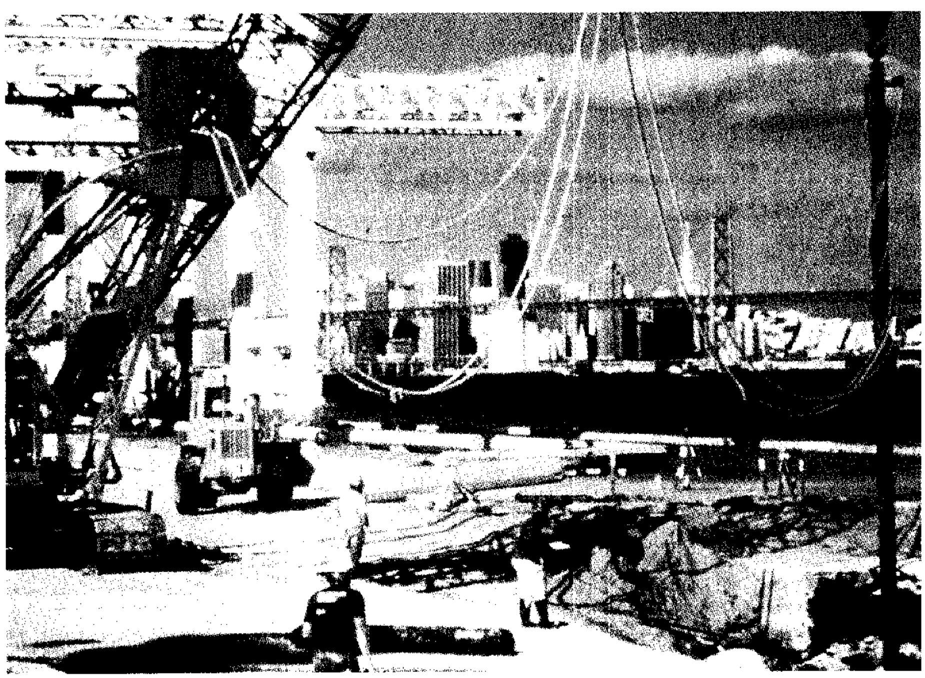

Figure 23. Construction of stone columns, Port of Oakland.

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 24 where the open circles represent the initial conditions.

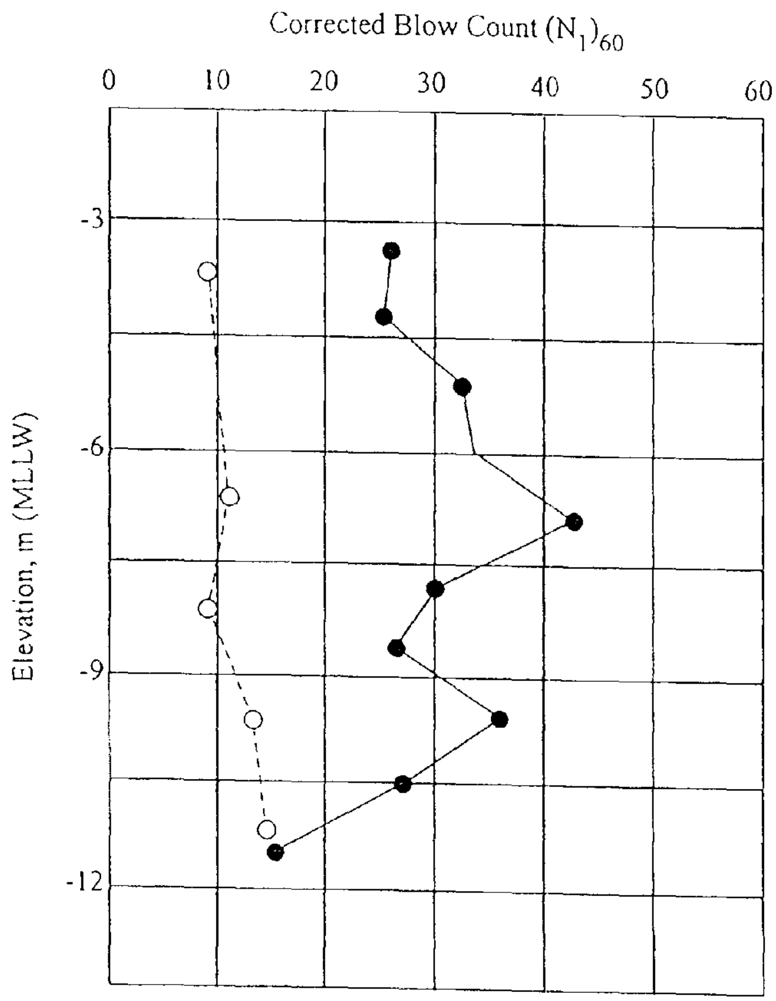

Figure 24. Standard Penetration Test resistance values for Port of Oakland wharf repair.

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. Underlying the loose, liquefiable sands and extending to depths of 24 m was medium-dense-to-dense sand containing greater amounts of silt and clayey silt. Groundwater was located at depths of 1.5 to 2.4 m.

The bridge, a 57.3-m-long by 28.3-m-wide single-span, post-tensioned, concrete box girder, is supported by 8.2-m-tall abutments at each end. Tapered retaining walls project at 45 degrees from each abutment end to support new embankments. The four wing walls,each approximately 30.5 m long, range in height from 0.6 m near the toe of the embankments to 8.2 m.

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. With the stone column support, shallow spread footings were designed with 216 kPa bearing pressure.

The vibro treatment was provided at each bridge end, as shown in figure 25, effectively treating an area measuring 38.4 m by 42.7 m 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 was used in order to penetrate a 1.5-m-thick working platform and extend to the bottom of the 4.6-m-thick 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 30 cm or an average per test hole of 98 blows per meter. A plate load test was allowed as an alternative test procedure to verify settlement reduction (figure 26).

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 testing.

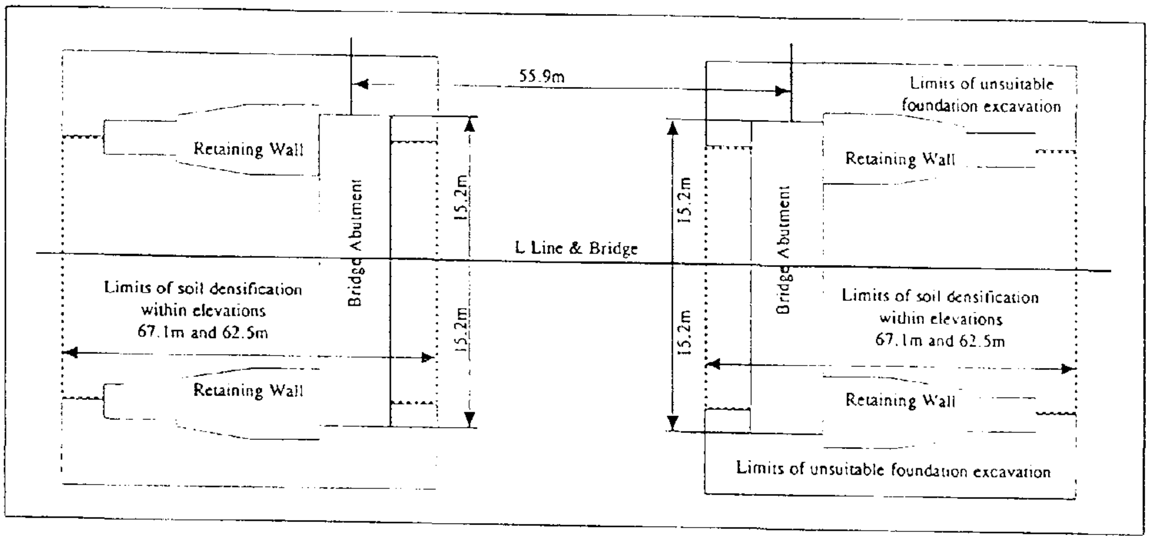

Figure 25. Vibro-replacement provided at each bridge end for Andresen Road.

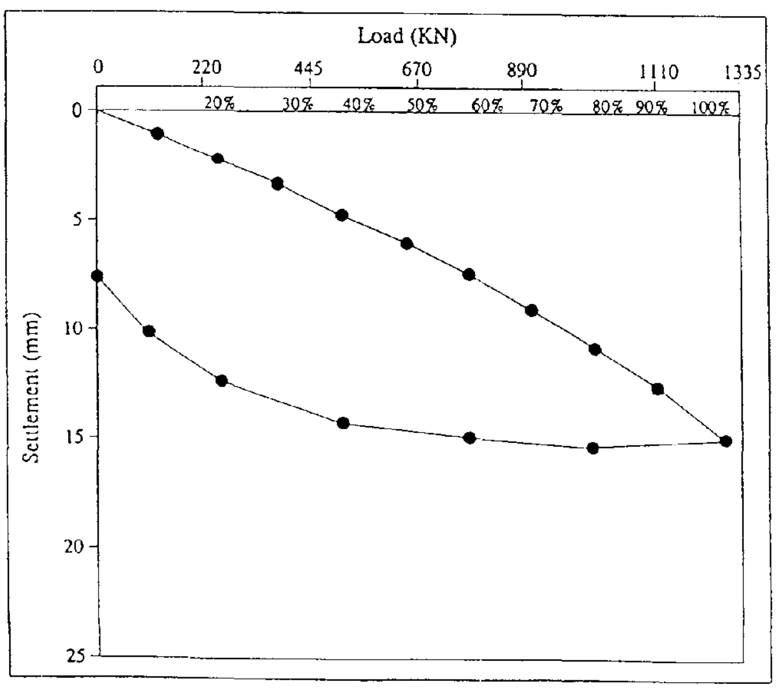

Figure 26. Plate load test to verify settlement reduction of Andresen Road.

REFERENCES

1. Barkdale, R.D. and Bachus, R.C. (1983). "Design and Construction of Stone Columns--Vol 1," *Report No FHWA/RD-83/O2C*. Federal Highway Administration. Washington, D.C.

2. Esrig, M.I. and Bachus, R.C. (1991). "Deep Foundation Improvements: Design, Construction, and Testing." *Proceedings of the ASTM Symposium on Design, Construction, and Testing of Deep Foundation Improvements: Stone Columns and Other Related Techniques*. ASTM, STP 1089, Philadelphia, PA.

3. Schaefer, V.R. (Editor) (1997). "Ground Improvement, Ground Reinforcement, Ground Treatment: Developments 1987-1997," *Proceeding of the ASCE Soil Improvement and Geosynthetics Committee Geo-Logan 97,* ASCE, NY, NY.

4. Hryciw, R.D. (Editor) (1995), "Soil Improvement for Earthquake Hazard Mitigation," *ASCE Geotechnical Special Publication No. 49,* ASCE, NY, NY.

5. Bachus, R. C. (1987) "Lessons Learned from European Practice on the Use of Stone Columns." *Prepared for 38th Annual Highway Geology Symposium*, Pittsburgh, PA.

6. Dobson, T. (1987). "Case Histories of the Vibro Systems to Minimize the Risk of Liquefaction." *Soil Improvement--A Ten Year Update. Geotechnical Special Publication No. 12,* ASCE, New York, NY. Ed. J.P. Welsh. pp 167-183.

7. Koelling, M.A. (1995). "Construction on Peat and Soft Soils." *Proceedings of the 9th Annual Symposium of the Canadian Geotechnical Society*, Vancouver, British Columbia.

8. Sung, J.T. and Ramsey, I.S. (1988). "Slope Stabilization by Stone at Wadhams, NY," *Report by Soil Mechanics Bureau*, New York State D.O.T., State Campus, Albany, NY.

9. Pengelly, A., Hayward Baker Inc. (1995), Personal Communication.

10. Venema, T.P., Waletzko, C.R and Swekosky, F.J. (1991). "Ground Modification for a 14-Story Structure Over a Deep Peat Deposit." *Foundation Engineering: Current Principles and Practices--Vol. 1.* Edited by F.H. Kulhawy, ASCE, New York, N.Y. pp 392-405.

11. Priebe, H.J. (1989). "The Prevention of Liquefaction by Vibro-Replacement." *Proceedings from Earthquake Resistance and Design.* Berlin, Germany.

12. Baez, J.I. and Martin, G.R. (1992). "Quantitative Evaluation of Stone Column Techniques for Earthquake Liquefaction Mitigation." *Proceedings of the Tenth World Conference on Earthquake Engineering*, A.A. Balkema, Brookfield, VT. pp 1477-1483.

13. Fox, S.N. Cowell, M.J. (1998). "Geopier™." *Foundation and Soil Reinforcement Manual.* Geopier Foundation Co., Scottsdale, AZ.

14. Fox, N.S. and Lawton, E.C. (1993), "Short Aggregate Piers and Methods and Apparatus for Producing Same," U.S. Patent No. 5,249,892.

15. Lawton, E.C., Fox, N.S. and Handy, R.L. (1994), "Control of Settlement and Uplift of Structures using Short Aggregate Piers," *In-Situ Deep Soil Improvement, ASCE Geotechnical Special Publication No. 45*, NY, NY, pages 121-132.

16. Priebe, H.J. (1995), "The Design of Vibro Replacement," *Ground Engineering, Volume 28, No. 10*, London, U.K., pages 31-37.

17. Cowell, M. and Fox, N. (1999). Personal Communication.

18. Munfakh, G.A., Abramson, L.W., Barksdale, R.D. and Juran, I. (1987). "In-Situ Ground Reinforcement." *Soil Improvement - A Ten Year Update*, ASCE Special Publication No. 12, NY, NY, page 331.

19. Goughnour, R.R. and Bayuk, A.A. (1979), "A Field Study of Long Term Settlements of Loads Supported by Stone Columns in Soft Ground", *Proceedings International Conference on Soil Reinforcement*, Paris, Vol. I, 279-285.

20. Colleselli, F., Mazzucato, A., Previatello, P., and Spalatro, A. (1983), "Improvement of Soil Foundation by Vibratory Methods," *Eight European Conference on Soil Mechanics and Foundation Engineering*, Helsinki, 223-228.

21. Bachus, R.C., and Barksdale, R.D. (1984), "The Behavior of Foundations Supported by Clay Stabilized by Stone Columns," *Eight European Conference on Soil Mechanics and Foundation Engineering*, Helsinki, 199-204.

22. Wallays, M., Dalapierre, J., and Van Den Poel, J. (1983), "Load Transfer Mechanism in Soil Reinforced by Stone or Sand Columns," *Eight European Conference on Soil Mechanics and Foundation Engineering*, Helsinki, 313-317.

23. Munfakh, G.A. (1984), "Soil Reinforcement by Stone Columns - Varied Case Applications," *International Conference on In-Situ Soil and Rock Reinforcement*, Paris, 157-162.

24. Greenwood, D.A., and Kirsch, K. (1984), "Specialist Ground Treatment by Vibratory and Dynamic Methods," *State-of-the-Art Report, Piling and Ground Treatment*, Thomas Telford Ltd., London, 17-45.

25. Seed, H.B. and Booker, J.R. (1976). "Stabilization of Potentially Liquefiable Sand Deposits Using Gravel Drain Systems." *Report No. EERC 75-10*, University of California, Berkeley.

26. Stack, T.D. and Yacyshyn, B.M. (1991), "Specifications for Constructing and Load Testing Stone Columns in Clays," *Deep Foundation Improvements: Design, Construction, and Testing ASTM STP 1089*, Philadelphia, PA, pp. 73-84.

27. __________ (1987), "Stone Column Inspection Guide for a Bottom Feed Vibro-Displacement Method." *Route 22, Wadhams MPM 22-1209-1167*, Essex County.

28. Egan, J.A., Hayden, R.F., Scheibel, L.L, Otis, M., and Serventi, G.M. (1992). "Seismic Repair at Seventh Street Marine Terminal." *Proceedings of the Conference on Grouting, Soil Improvement and Geosynthetics--Vol 2.* Geotechnical Special Publication No. 30. ASCE, New York, NY. pp 867-878.

SOIL MIXING

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CHAPTER 1

DESCRIPTION AND HISTORY

Soil mixing for deep soil stabilization comprises ground improvement techniques which mix reagents with soils at depth to improve soil properties in situ, without excavation or removal. The mechanical mixing aspects of reagents with in situ soils distinguishes this ground improvement method from others. These methods of soil improvement can be utilized for ground water cut-off, excavation support, soil stabilization, settlement reduction, foundation support, and the fixation of contaminated soils. Surface mixing with stabilizers for pavement subgrade improvement is considered outside the scope of this technical summary.

Deep soil stabilization 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 identified categories which fall in two distinct groups as follows:

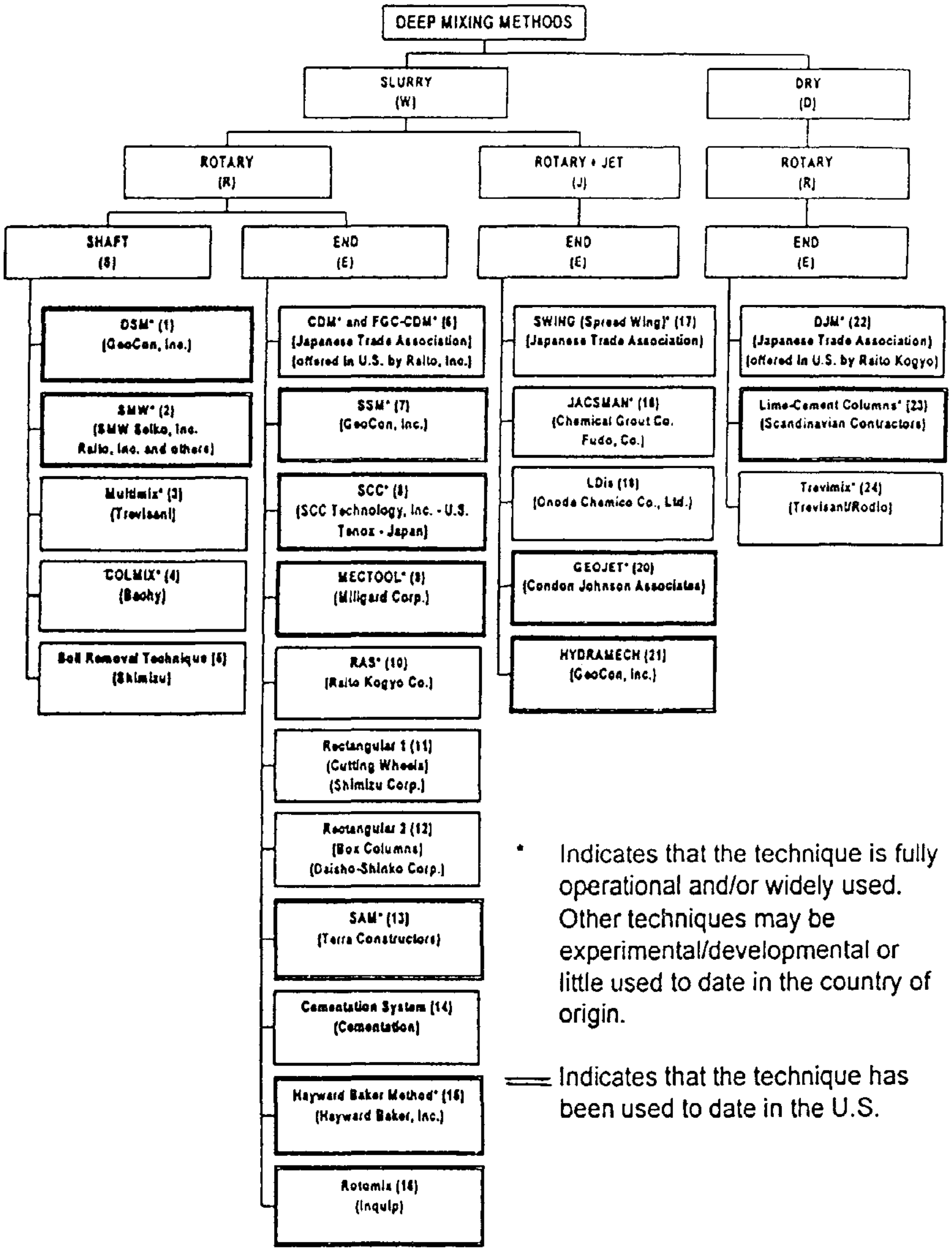

Figure 1. Generic classification of Soil Mixing techniques.

Deep Mixing Methods (WRS, WRE, WJE)	***refers to wet***, single or multi-auger, block or wall techniques developed for large scale foundation improvement and containment in any soil. Primary reagents are cement-based grouts.
Lime and Lime-Cement Columns (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. This technique was developed in the United States in 1956 by Intrusion Prepakt in Cleveland, Ohio. 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 water tight cut-off walls. This system was successfully used for cut-off walls beneath dams, beach front 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 were obtained. In silt or silty sand, strength was less than 7 MPa. 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 increase, 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 .[(3)]

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 and 4 million meters per year.[(1)]

In the last five years 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 became 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 tunnelling, cutoff walls and as a stability buttress.

Lime column technology has been discussed in United States technical literature since 1977. However, the first lime column system was not installed until 1992 as part of a research effort by the Florida Institute of Phosphate Research to determine if this technique could be of use in recovering an estimated 35 hectares of 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 using primarily lime-cement, and cement as a reagent. 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 result in the current absence of a comprehensive technical reference and cost information.

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]

The most comprehensive review of available equipment characteristics and treated soil properties obtained by in situ mixing methods is contained in Bruce "An Introduction to the Deep Soil Mixing Method used in Geotechnical Applications, Verification and Properties of Treated Ground," FHWA RD-99-167.[1]

CHAPTER 2

DESIGN CONSIDERATIONS

As with other emerging technologies, the applications of deep soil stabilization continues to expand with every completed project. This chapter discusses deep soil stabilization applications, advantages and disadvantages/limitations, applicable soil types, the steps required to develop a feasibility evaluation, as well as addressing competing conventional technologies.

2.1 APPLICATIONS

Wet Deep Mixing Methods 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.

a. Wet Mixing

Deep mixing methods (DMM) using wet reagents were originally developed for sea reclamation and were subsequently used for land-based applications. Figure 2 gives an overview of typical ocean and land applications. Five main applications have been identified.[(3)]

Foundation improvement

DMM is used extensively in Japan for large-scale underwater soil improvement to provide a stable foundation capable of bearing the structural loads. Between 1960 and 1980, some 600 km^2 were reclaimed from the sea for industrial and public use.[(3)] 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 project in Boston, MA.

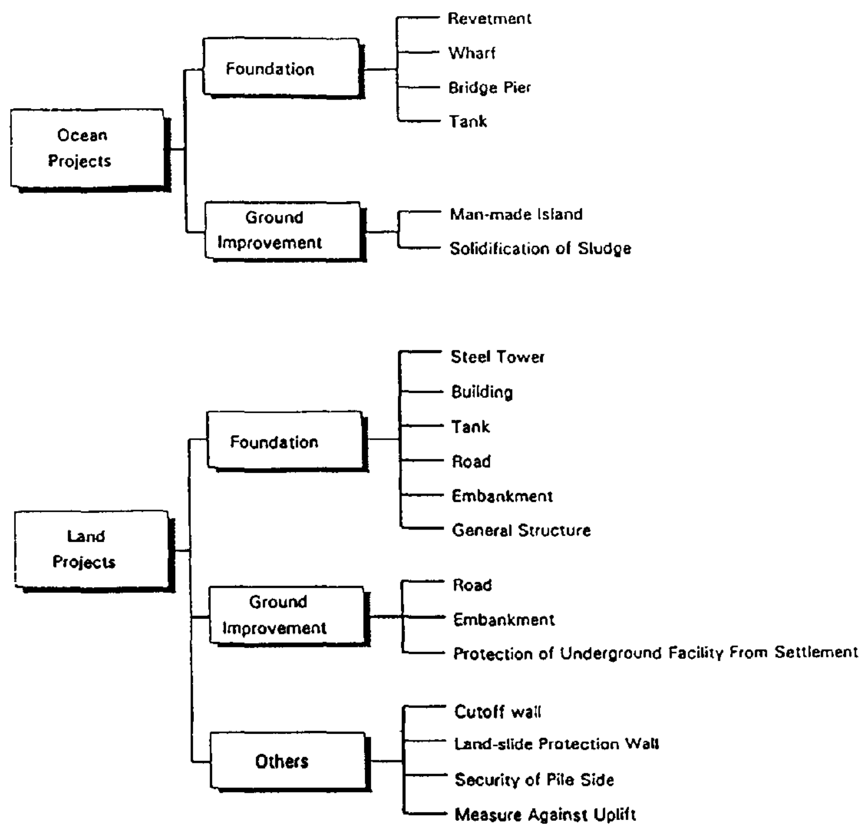

Figure 2. Range of applications for deep soil mixing.

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 ground water infiltration. 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 has been made in connection with excavation support in Wisconsin at Lake Parkway and more recently on a number of projects at the Central Artery in Boston, Mass 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 to serve as ground water 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 sludges 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:

- 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 were 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:

Settlement control

Both total and the 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 as for stone columns, 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 that for wick drain projects, provided that the permeability of the column is at least one order of magnitude less than the native soil. Lime columns can meet this 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. Alternatives 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 for marine work and up to 30 m 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.

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 and will not generate noise and vibration. It is an ideal construction method for use 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. Further, the strength of DMM is time dependent which may require laboratory testing times up to a few months although a 28-day strength is the usual strength index.

Subsurface utilities and lifelines must be relocated in order for DMM to be performed. Rigs require substantial headroom. This is a limitation for lime-cement columns as well.

Although the quantity of waste is generally less than for some other technologies, DMM may produce between 30 and 100 percent of spoil soil volume, depending on project specifics, equipment, methods used and in situ water content. Disposal of the waste is a limitation of the technology.

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 of the ground water 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 when 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 includes the discharge of limited to no waste material to the ground surface, which eliminates disposal costs which may be costly if hazardous. The use of cement columns as developed in Japan has not been demonstrated to date in the United States on transportation projects.

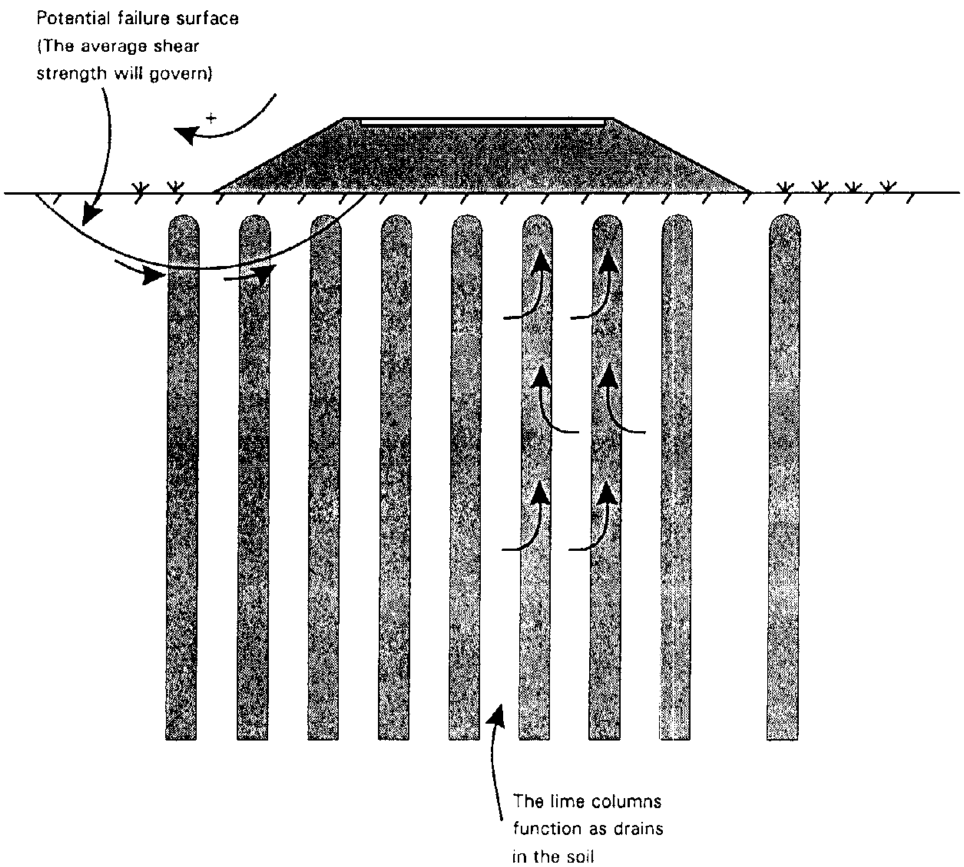

Figure 3. Lime columns installed for a road embankment.

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 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, or for design consider a reduced strength (28-day) to compensate, as creep strength is related to unconfined compressive strength.

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.

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.

2.3 FEASIBILITY EVALUATION

The first step on any project is a subsurface investigation. 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 involves 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, as may occur in the case of lifelines, then an alternative solution will be required in that location. With these soil stabilization techniques, a significant percentage of the volume of treated soil may be displaced, requiring adequate on-site storage prior to off-site disposal.

The design site investigation should include:

- Evaluation of soil type - governing soil type; existence of any type of obstruction; existence and level of organics.
- Water content in situ.
- Engineering property determinations, strength, compressibility, classification index properties.
- pH.
- Chemical and mineralogical properties to assess presence of pozzolanic materials including soluble silica and alumina which affects lime reactivity only.
- Ground water 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. Further there should be unrestricted overhead clearance, and a need for a relatively vibration free technology to improve large volumes of soft soil. 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 to 2 MPa, but more typically 0.6 MPa, can be achieved.(1) With wet mixing techniques presently available in the United States (1999) the reported range is much wider, 0.2 to 10 MPa, but more typically in soft soils in the range of 0.2 to 3 MPa.(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 property values is a function of the in situ soils, the equipment used for mixing and the water content in situ.

Table 1. Typical improved engineering characteristics.

Property	Typical Range
Unconfined Compressive Strength q_u	0.2 to 5.0 MPa granular soils 0.2 to 3.0 MPa cohesive soils
Permeability (k)	10^{-4} to 10^{-7} cm/s
Young's Modulus (E_{50}) (Secant Modulus at 50% q_u)	50 to 1000 q_u for lab samples (Japan) 50 to 150 q_u for lab samples (US) 100 to 300 q_u (typical range)
Tensile Strength	8 to 14 percent of q_u

It should be noted that certain soil chemistry factors significantly impede soil stabilization with cement. Soil chemistry limitations are summarized on table 2.

Table 2. Favorable soil-chemistry factors.

Property	Favorable Soil Chemistry
pH	Should be >5
Natural water content w_c	Should be less than 200%
Organic Content	Should be less than 6% (Wet method)
Loss on Ignition	Should be less than 10%
Humus Content	Should be less than 0.8%
Conductivity	Should be greater than 0.4 mS/cm

For soils outside these limits, stabilization is possible at lower strength levels by the addition of certain stabilizers and special additives.

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 should be considered even if a performance type of specification is used for construction. The key physical and engineering properties of the composite material will depend on the type and percentage of the reagents utilized and the character of the in situ soil. In the design of a 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 conducted as a function of time.

These preliminary laboratory testing results should 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, rate of penetration and withdrawal cannot be modeled by the laboratory testing program. Further, note that for significant stabilization to occur a minimum cement content of 5 percent 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 and can be used constructively on the site.

Dry mix methods are most advantageous where the in situ silts or clays have a 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 yield higher strengths and significant spoil quantities.

Optimum results are obtained where the in situ water content of the clays and silts are at least 60 percent.

The key issue to be resolved for site remediation applications are what type of chemical or other reagent should be injected and mixed into the contaminated soil, and in what quantities should they be injected. The answer to both questions is 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 Mix and Wet Mix 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 has 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 which 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]

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.

- 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.

Much recent research and interest has been directed towards developing indicators of the potential efficiency of the mixing process which 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 characterics as follows:

$$T = N\left(\frac{Rp}{Sp} \cdot \frac{Wi}{W} + \frac{Rw}{Sw}\right)$$

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	=	stablizer 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 SMW Seiko 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 to 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 to 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* Materials type and strength *requirements)* w/c ratio Cement ratio (k_{cement}/m^3_{soil}) Volume ratio ($Vol_{grout}:Vol_{soil}$)	 Cement grout ± bentonite and other additives w/c = 1.2-1.75 120-400kg/m^3 soil 15-40%	 Cement grout ± 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. k E	 0.3-7 MPa (clay strengths approx. 40% those in sands) 1 x 10^{-7}- 1 x 10^{-9} m/s 300 to 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 to 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.A.): Tenox (Japan)	WRE HBM Hayward Baker Inc.: A. Keller Co. (U.S.A. 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	Cable suspended shaft rotated by bottom rotary drive table. 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	Single with 3 rotated mixing blades plus "share blade"	Single with 2 or 3 pairs of paddles above drill bit
Diameter	0.6-1.5m;1.2m double shafts	0.5-3.5m, typically 2.1m to 2.4m
Depth	20m max	20m max
RPM	30-60	20-25 (penetration): higher upon withdrawal
Productivity/output	1 m/min in and out 400m of piles/shift	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	Typically cement, but others e.g., ash, bentonite possible.	Varied in response to soil type and needs
w/c ratio	0.6 (clays) to 1.2 (sands)	1-2
Cement ratio (k_{cement}/m^3_{soil})	100-450 kg/m^3 cement	150 kg/m^3 cement
Volume ratio ($Vol_{grout}:Vol_{soil}$)	30-35%	15-30%
Reported Treated Soil Properties		
U.C.S.	3.5-7 MPa (sands) 1.3-7 MPa (cohesives)	3.5-10 MPa (sands) 0.2-1.4 MPa (clays)
k	10^{-8} m/s	10^{-10} m/s
E	180 UCS	
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.A.	Developed since 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.A.	DRE Lime-Cement Columns Various (Scandinavia and Far East); Stabilator (U.S.A.) Scandinavia, Far East, U.S.A.
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	1 shaft with pair or wings	Single shaft, various types of cutting/mixing blades.
Diameter	0.6-1.2m	500-1200mm, typically 600, 800mm
Depth	45m max (25m typical)	25m max
RPM	150-200 (recent developments focusing on slower rpm)	
Productivity/output	2-12 m/min (penetration) 15 m/min (withdrawal) 150m of piles/hr possible	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	Cement, additives	Cement and lime in various percentages (typically 50:50 or 75:25)
w/c ratio	0.5-1.5 (1.0 typical)	23-28 kg/m (600mm diameter);
Cement ratio (k_{cement}/m^3_{soil})	150-300 kg/m^3	40 kg/m (800mm diameter) i.e., 80-150 kg/m^3
Volume ratio (Vol_{grout}:Vol_{soil})	20-40%	
Reported Treated Soil Properties U.C.S.	0.7-5.5 MPa Bay mud 4.8-10.3 MPa (clay)	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 since 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) Taisei Corporation/Raito Kogyo, Co. and others. 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; 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 40 m N/A 0.03-0.1m/min penetration
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 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. k E	Up to 10 MPa Up to 1 x 10^{-9} m/s 100 to 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 from 1998.	SWING Association with 17 members established in late 1980s in Japan.

Table 3 (cont'd). Details of operational soil mixing techniques.[1]

Classification **Name** **Company** **Geography**	WRE Single Auger Mixing (SAM) Terra Constructors U.S.	WRE MecTool® Millgard Corporation U.S. and U.K.
General Description of Method	Large-diameter mixing tool on hanging leads rotated, with slurry injection during penetration.	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	Multiple-auger mixing capability (MAM) foreseen for deeper applications.	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	 1 1-3.6 m 13 m max. 8-16 380 m^3/8-h shift	 1 1.2-4.2m max. 25 m max (typically less than 6m) N/A 0.6 m/min.
Mix Design *(depends on soil type and strength requirements)* Materials	Cement grout mainly, and other additives for oxidation/stabilization of contaminants.	Cement grouts including PFA and other materials ± proprietary additive to breakdown "plastic seals thereby enabling through-the-tool delivery"
w/c ratio	0.75-1.0	N/A
Cement ratio (k_{cement}/m^3_{soil})	N/A	N/A
Volume ratio ($Vol_{grout}:Vol_{soil}$)	10-20% by weight	20-35% estimated range
Reported Treated Soil Properties U.C.S.	Varies dependent upon soil type; up to 3.5 MPa	0.8-2.5 MPa
k	Similar to in situ soil	1×10^{-8} to 1×10^{-9} m/s
E	N/A	N/A
Specific Relative Advantages and Disadvantages	Applicable in soils below water table. Environmental applications.	Excellent control of grout and spoil quantity.
Notes	Developed since 1995.	Mainly environmental applications to date.

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- to 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 ± 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	1	1-3, typically 3. Configuration varies with soil.
Diameter	1-4m	0.55-0.8 m at 0.4 to 0.6-m spacings
Depth	12 m	25m
RPM	15	12-30
Productivity/output	500-1500 m^3 per shift	0.35-1.1 m/min penetration (typically 0.5) 0.48-2 m/min withdrawal
Mix Design		
(depends on soil type and strength requirements) Materials	Cement grout, bentonite, flyash, lime, and other additives for contaminant immobilization	Cement grout mainly, plus bentonite in sands; additives common, even in predrilling phase
w/c ratio	1-1.75	Typically low, i.e., 0.6-1.0 (especially in cohesives)
Cement ratio (k_{cement}/m^3_{soil})	200-400 kg/m^3	200-250 kg/m^3 typical (80-450 kg/m^3 range)
Volume ratio (Vol_{grout}:Vol_{soil})	12-20%	15-40%
Reported Treated Soil Properties U.C.S.	3.5-10 MPa in granular soils. 0.6-1.2 MPa common in high-water-content sludges.	0.5-5 MPa (sands); 0.2-1 MPa (silts, clays); up to 20 MPa in very hard soils.
k	1×10^{-10} m/s possible.	$<1 \times 10^{-8}$ m/s
E	100 to 300 x U.C.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.

Table 3 (cont'd). Details of operational soil mixing techniques.[1]

Classification **Name** **Company** **Geography**	DRE Trevimix TREVI, Italy Italy, Eastern 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) 30 m 10-40 0.4 m/min penetration 0.6 m/min withdrawal 39 m/8-h shift
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})	 Dry cement (most common), lime, max. grain size 5 mm N/A 150-300 kg/m^3 N/A
Reported Treated Soil Properties U.C.S. k E	1.8-4.2 MPa (avg. 2.5 MPa) N/A 1 to 2.66 x 10^3 MPa (clays) 3.125 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.

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 additive procedures. The following discussion is in generic terms to provide an overview of typical construction methods and equipment.

a. Wet Deep Mixing Methods (DMM)

DMM is performed by individual or multiple-stem mix tools, either barge-mounted for off-shore work, as shown in the example illustrated in figure 4, or based on tracked cranes for work on land, as shown in figure 5. Multi auger equipment is shown on figure 6.

For off-shore 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 m^2 to 9.5 m^2, with the maximum depth of treatment being up to 70 m from the water surface.[3]

For on-shore operations, the soil mixing equipment is typically supported by a conventional, track-mounted crane. 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 as compared to up to 70 m.[1] The mixing plant generally consists of a slurry

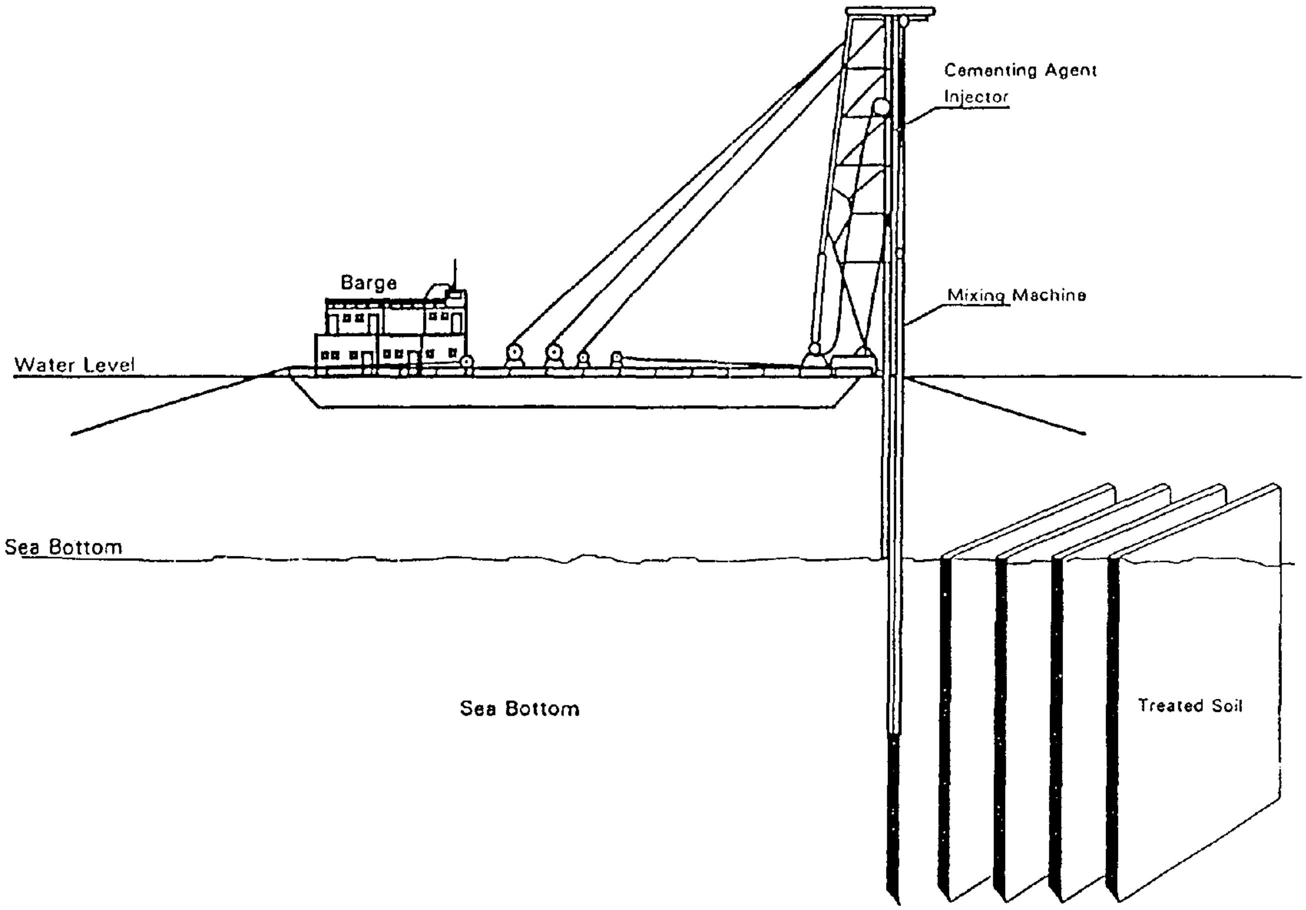

Figure 4. Schematic of barge mounted deep soil mixing rig.

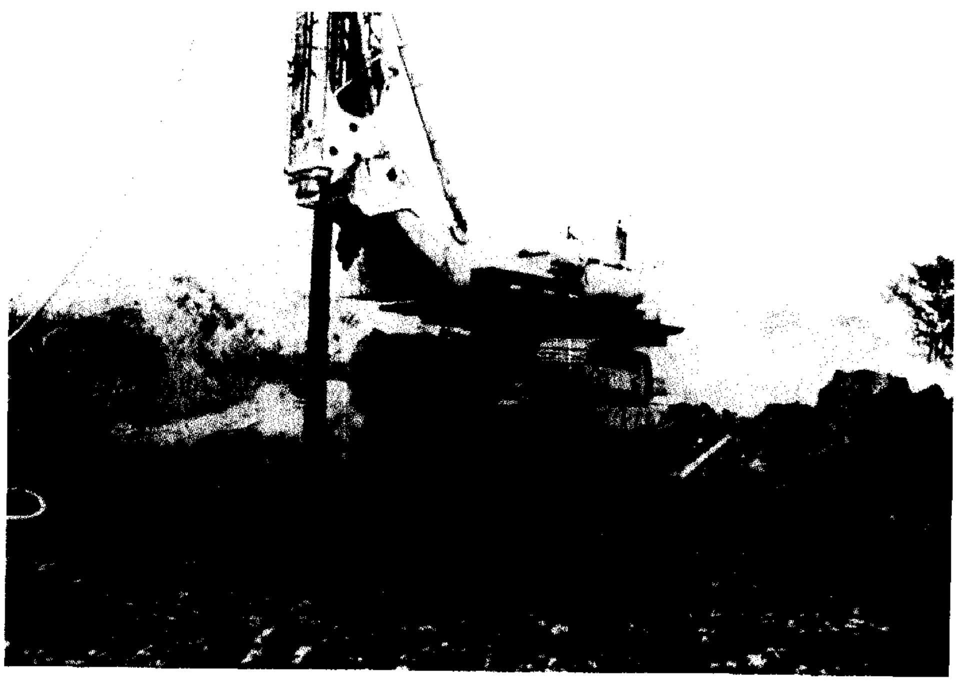

Figure 5. Track-mounted equipment.

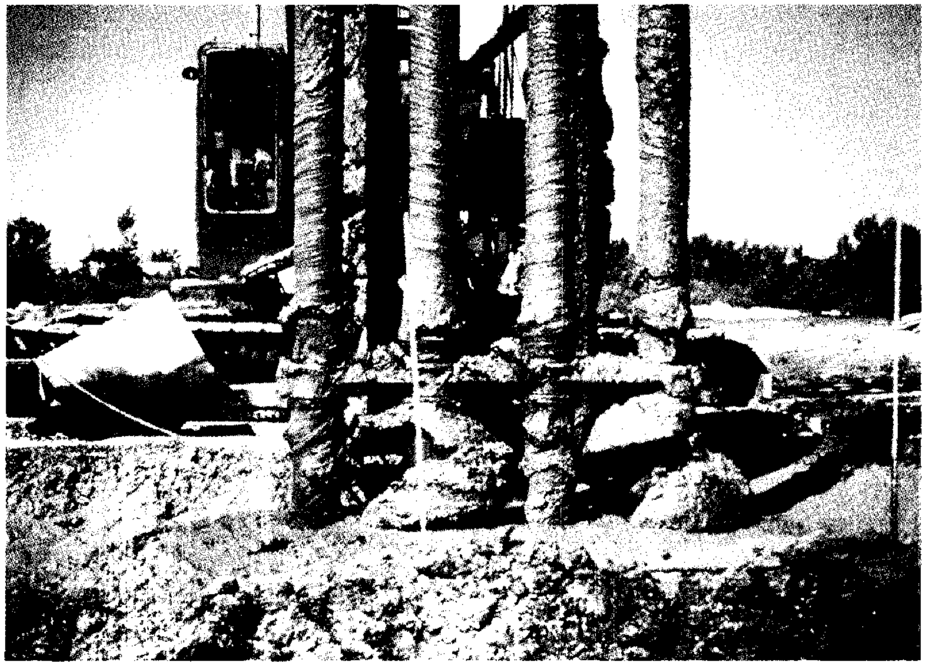

Figure 6. Multi auger mixing equipment.

mixer, slurry agitator, batching system, slurry pumps and a computer for mixing and slurry control. An automated batching system may be used that measures the water, cement and other additives by weight to produce a more consistent slurry. The desired weight of each slurry component can be preset and 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 off-shore 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 provide 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 pugmill (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 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 and 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 soils 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 soil lumps increase. For highly plastic clays, part of the clay cannot be broken down during the soil mixing process and remain 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.

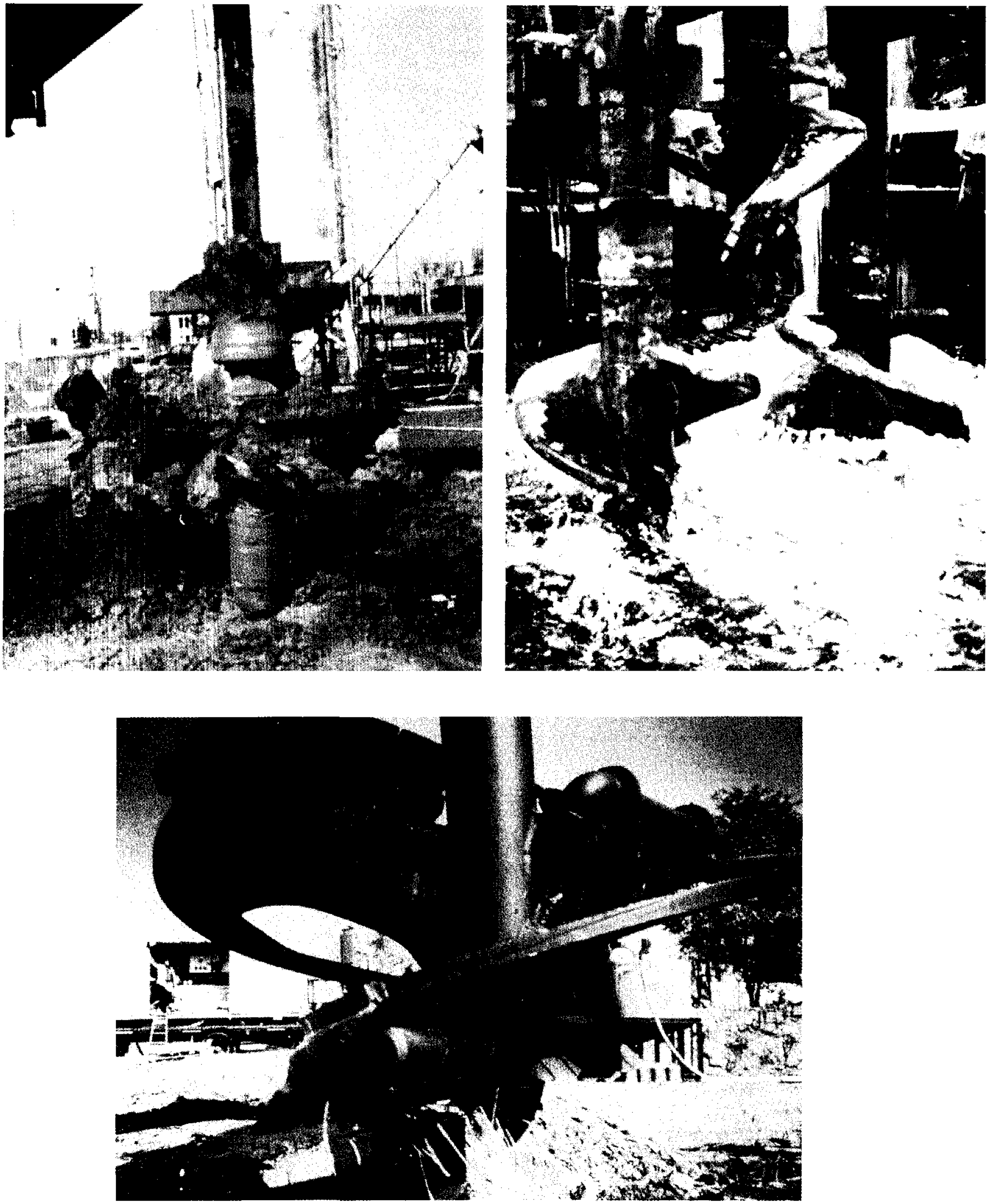

Figure 7. Various mixing tools and blade orientations.

Shallow soil mixing can be accomplished with a single, large diameter (from 1.0 to 3.7 m) 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. This kelly bar is powered by a turntable capable of producing a torque of up to 406 kN-m. The single auger system is typically limited to 15 m 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 of soil per 8 hour shift.

Figure 8. Shrouded boring/mixing tool.

b. Dry Mixing, Lime-Cement Columns (LCC)

LCC 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 on 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 length is at present 30 m. 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.

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.

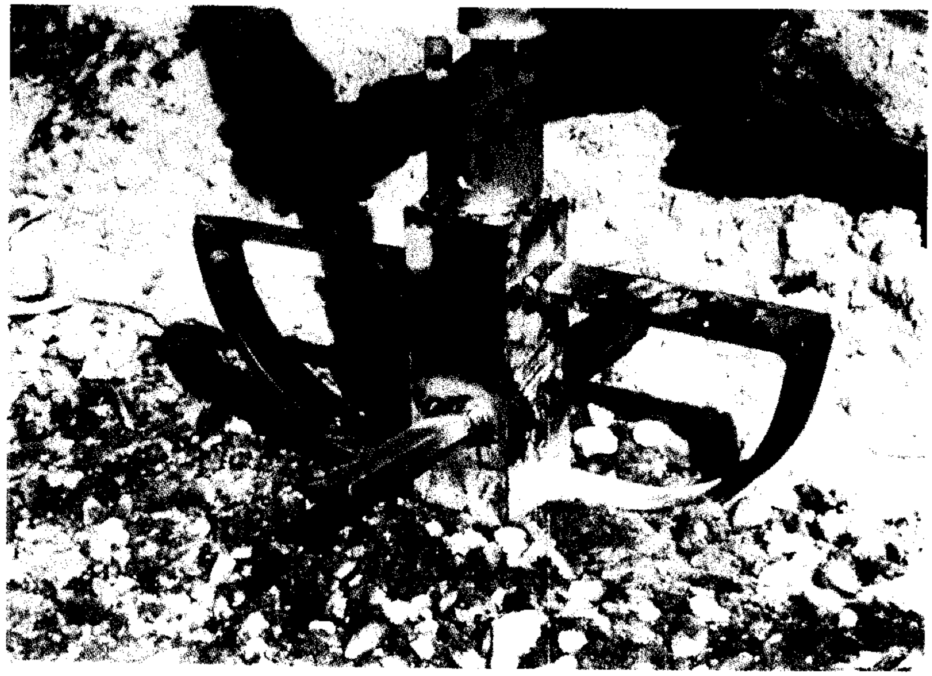

Figure 10. Mixing tool for lime-cement.

3.2 MATERIALS

DMM was originally developed with cement as the primary stabilizing agent and LCC technology developed using unslaked lime. At present, lime-cement is used most of the time with the lime proportion varying from 15 to 40 percent in the United States. For liquefaction mitigation 100 percent cement is used for stabilizing sands. 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 (DMM)

Ordinary portland cement is typically used in the DMM process. The cement is typically introduced as a 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, minerology, organic content, pH and water content.
- Reagent content, type, quality, mixing water and additives.
- The degree of mixing, penetration and withdrawal speed.
- 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 (LC, LCC and Cement Columns)

Lime Columns

Lime column construction uses unslaked lime. A rapid hydration reaction occurs when lime is added to wet soils and absorbs the pore water. The water content of the soil is reduced as the lime increases in volume, and this is accompanied by an increase in shear strength.

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 a 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

Lime-cement mixtures are almost exclusively used at present. The ratio of cement to lime varies depending on application and target engineering properties, but is often at a 75 to 25 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 ground water temperatures varying between 10° and 15° 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 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 is 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 to 1/3 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 the pozzolanicity may be evaluated by measurements of conductivity. The following ranges of conductivity may be used as indicators:

Conductivity in mS/cm	Pozzolanicity
<0.4	non-pozzolan
>0.4 <1.2	ordinary
>1.2	good

Typically high soil 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.

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 with other reagents for treating soils with heavy metals and sludges.

- Dispersant additives 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 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 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 to a project on 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 on figure 11, and equipment demonstrating a block type installation is shown on 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.

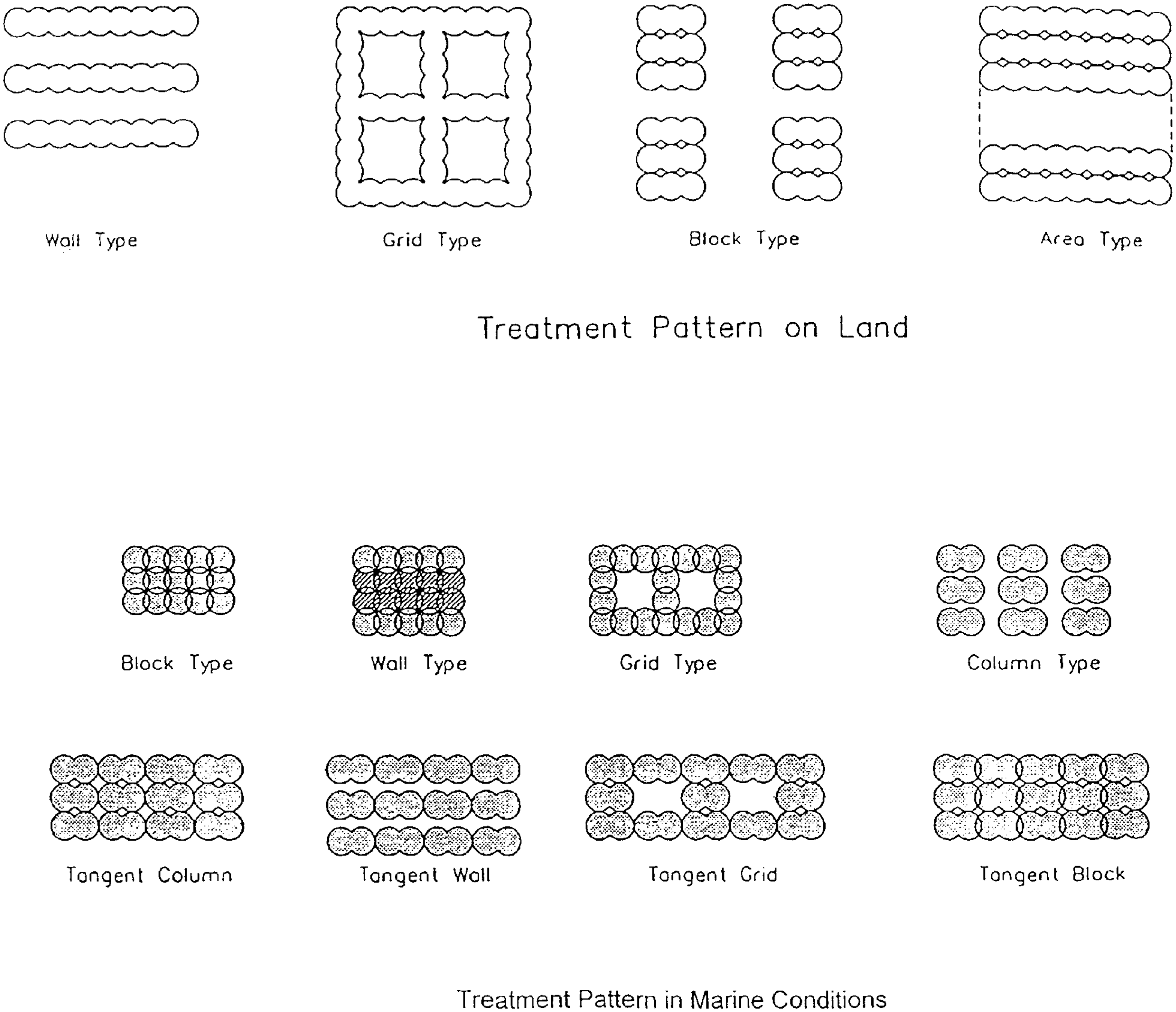

Figure 11. Basic deep mixing treatment patterns.[1]

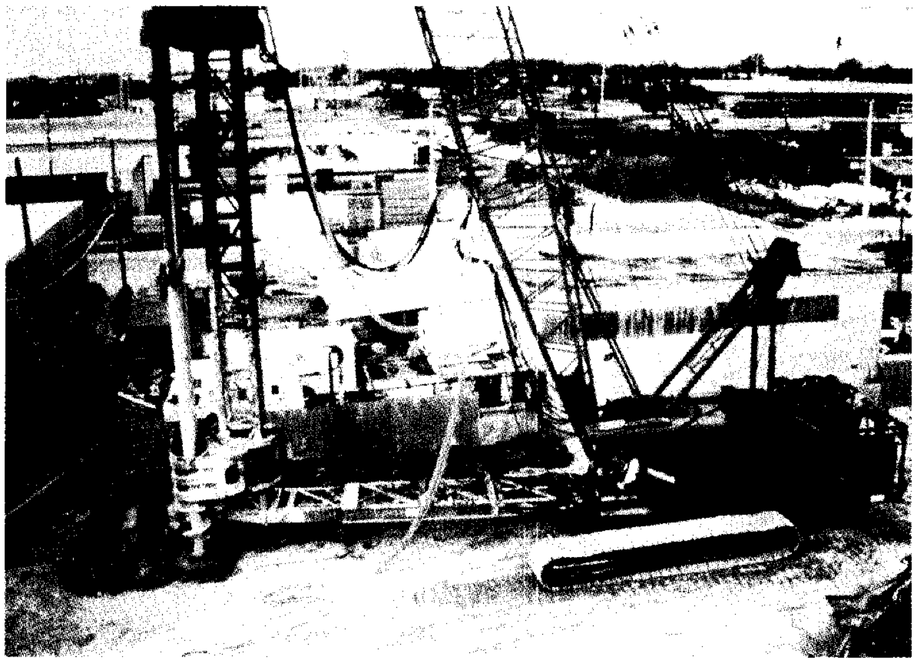

Figure 12. Equipment installing a block type pattern.

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. In the United States for highway applications, a significant use is expected to be for excavation support and groundwater control.

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 reinforcement members in lieu of lagging which it replaces.[10]

The analysis consists of determining the maximum bending moment and the maximum shear force, using the bending rigidity of the reinforcement member on a unit width. The minimum targeted compressive, shear strength and modulus for the soil cement is used in the analysis. The internal stress analysis include shear stress along vertical planes between two H-piles and compressive stress redistribution inside the soil cement facing. Tensile strength is generally not considered in the design. 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 on figure 13.[11]

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 $1\text{x}10^{-6}$ cm/sec or less is usually required for permanent seepage control in dam, dike or dry dock projects.

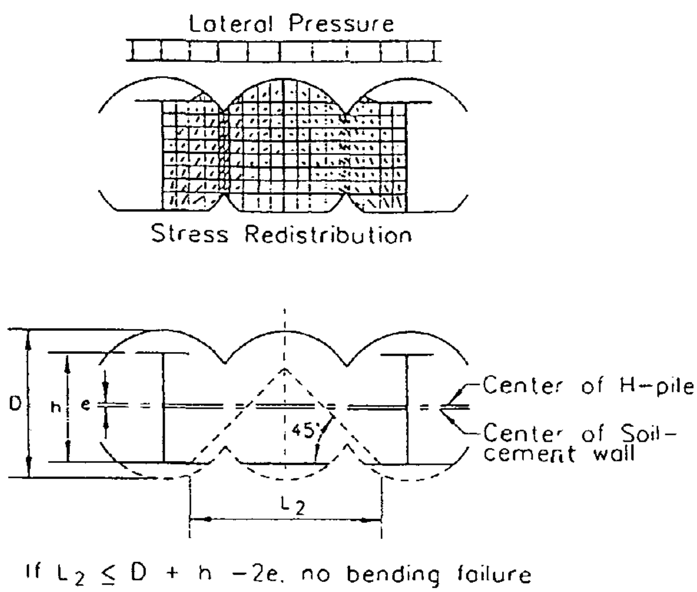

Figure 13. Stress analysis of soil-cement wall.[11]

b. Liquefaction mitigation

The use of soil mixing for liquefaction mitigation should provide confinement, some reinforcement, and some drainage.

For liquefaction prevention associated with at grade structures, a mixed soil perimeter cut-off wall is installed to isolate the loose, cohesionless soils beneath the structure. Some reinforcement is achieved by constructing mixed soil walls in block, wall or grid patterns and varying the spacing to provide some direct additional support during a seismic event and reduce settlement. 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.

c. Stabilization for structural support[3]

For large scale structural support, a continuous treated soil mass of the wall or area type configuration as shown on figure 11 is 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. 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.

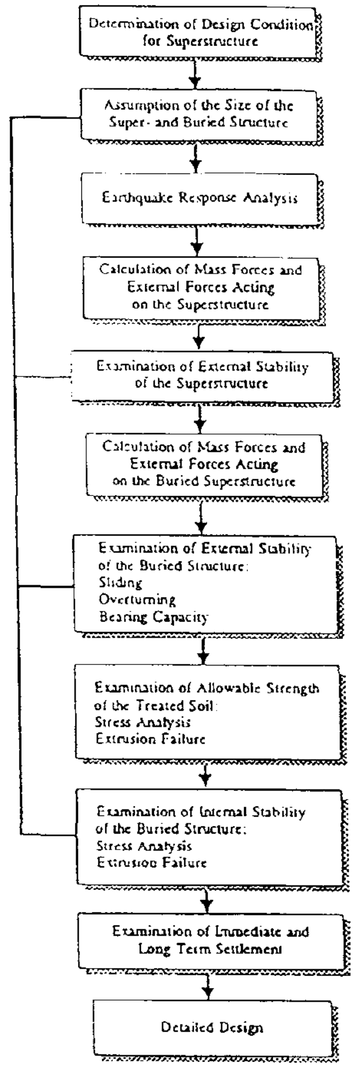

Figure 14. Flow chart for design of continuous soil mass.

- *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.

- *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.

d. Stabilization and/or settlement reduction for embankments

The design concept used in connection with applications for settlement reduction and/or stability enhancement are 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. 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 pattern of columns is used. For stabilization under embankments the columns are overlapped and constructed perpendicular to the centerline under the critical areas. The columns are often connected with shear walls.

The Area Replacement Ratio a_s, is defined as:

$$a_s = \frac{A_s}{A} \qquad (1)$$

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 the average applied vertical stress, σ_s the stress on the soil mixed column and σ_c the stress on the adjacent soil. For computations the following relationships are useful:

$$\sigma_s = \frac{nq}{[1 + (n-1)\,a_s]} \qquad \text{(on column)} \qquad (2)$$

$$\sigma_c = \frac{q}{[1 + (n-1)\,a_s]} \qquad \text{(on soil)} \qquad (3)$$

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:

$$\tau = \tau_f a_s + C_u\,(1 - a_s) \qquad (4)$$

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. The area ratio is the ratio of the area of one column to its tributary. 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.

The literature contains minimal data on measured stress ratios and their dependence. The stress ratio for a highway project in southeast China, using dry mix CC 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 and the soft foundation soils were characterized by water contents between 60 and 80 percent and undrained shear strengths from 15 to 30 kPa. The 28-day unconfined compressive strength of the soil mix columns was on the order 2000 kPa 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. Scandinavia 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 with a replacement ratio of 0.4. At this site the in situ clays are characterized by moisture contents of 30 to 45 percent and an undrained shear strength of 45 kPa. The LCC columns were designed for unconfined compressive strengths between 600 and 800 kPa. At a recent Virginia DOT test site stress ratios of 2.5 were measured even with a low area replacement ratio of 0.07.

It is anticipated that area replacement ratios of 0.20 to 0.35 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 reasonably dependent on area replacement ratios. 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 the columns being 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 spacings in excess of four diameters, the stress ratio will approach unity. Once the load distribution has been determined or assumed, settlement 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 column is somewhat greater than the adjoining soil, but with lime cement or cement 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 inhomogeneities/local overdosage 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.

e. Stability enhancement for trench support[2]

Lime or LCC can be used to stabilize excavated trenches when the depth is less than 4 m. The columns should overlap in order 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 as for a gravity retaining wall. Bracing should be sufficient to resist potential water pressure behind the wall.

The stability of the column wall, with respect to overturning, can be increased by constructing multiple parallel rows of columns so that they function as buttresses. Then, the unstabilized soil between the buttresses will contribute to the stability of the wall, with respect to overturning. The stability of the wall will then be governed by the average shear strength of the soil along a circular or a plane failure surface through the wall.

The number of columns is governed by the required factor of safety with respect to a shear failure surface through the wall.

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 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 tests 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 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 to 450 Kg/m^3 of soil. The relationship between cement dosage and unconfined compressive strength for Japanese soils is shown on figure 15. Note that Japanese clays typically are at a higher natural water content than clays in the United States.[11]

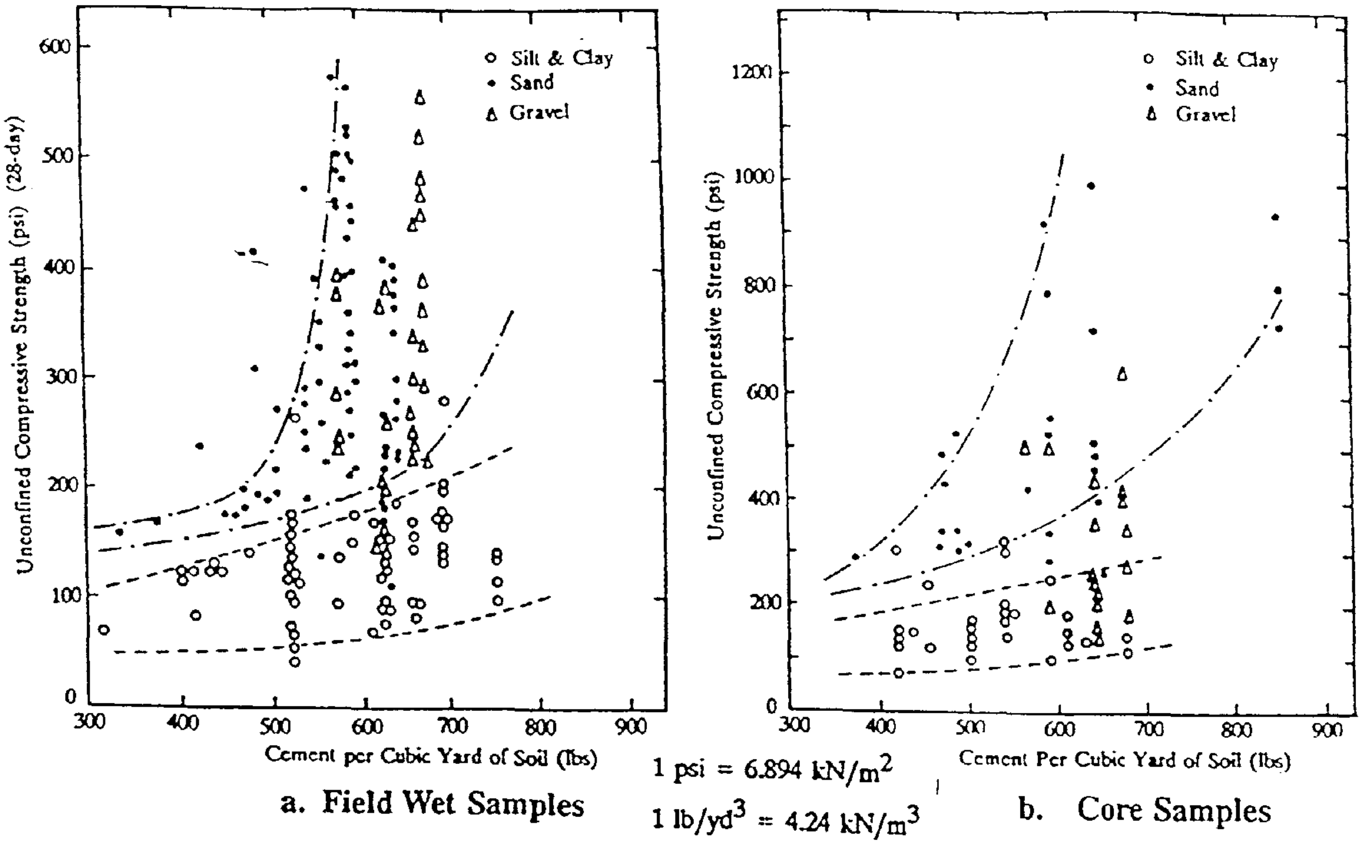

Figure 15. Strength of soil-cement.[11]

- *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 0.8 to 2 but are more typically between 0.8 and 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.

- *Variability of field strengths.* Significant variability in field strengths has been reported. The coefficient of variation from the mean has been reported in Japan to be in the range of 0.2 to 0.35, the higher variation typically associated with clay soils.[12] 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.

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 on 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
Soft clays	0.4 - 1.5 MPa
Medium/hard clays	0.7 - 2.5 MPa
Silts	1.0 - 3.0 MPa
Fine-medium sands	1.5 - 5.0 MPa

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 percent 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 and up to 50 percent 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]

- Poisson ratios vary from 0.3 to 0.45. A value of 0.26 is often used in Japanese design practice.[1]

- The soil mix material behaves as an 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 average field strength measured be greater than the design strength by at least 1.3 times one standard deviation developed from conformance (or lab) testing.

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 depend 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.

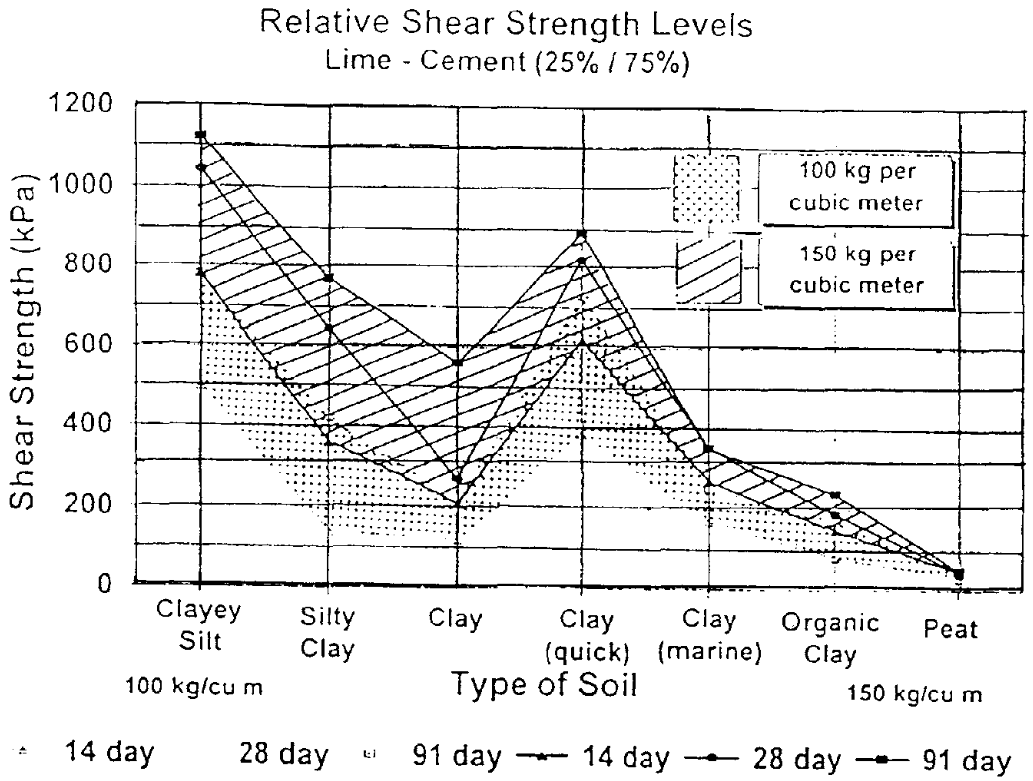

Figure 16. Shear strength of different soils mixed with two quantities of lime and cement at three curing times.[14]

Table 5. Typical improved engineering characteristics for lime-cement.

Property	Typical Range
Undrained shear strength C_u	10 to 50 *x* C_u of soil (150 to 1000 kPa)
Youngs Modulus	50 to 200 *x* C_u (LCC) 50 to 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 to 1000 times

Typically, 80 to 150 Kg/m^3 of reagent at a ratio of 75 cement, 25 lime or 15 percent lime and 85 percent cement is used. The 28-day unconfined compressive strength, typically used for design, will increase from 15 to 30 percent 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 preconsolidation 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 of 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 reduces improved soil strength.
- The compressibility of the improved soil is reduced as the preconsolidation pressure is increased and moisture content decreased.
- The plasticity index is reduced.
- 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 is typically used for design of LCC columns regardless of laboratory results. Currently, significant interest and research is focused in the use of dry soil mix methods for stabilizing peat and other highly organic soil deposits. The use of significantly larger percentages of cement and additives have resulted in a low strength stabilized soil that may be attractive for certain applications.

Key to any soil mix application is a design or pre-production phase laboratory testing program. 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 percent. The reported range of improved engineering characteristics may be similar 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, 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 tests:

- Laboratory testing of samples (before construction).

- Wet grab sampling of fluid in situ material (during construction).

- Coring of hardened in situ material (after construction).

- Exposure, and cutting of block samples (after construction).

- Geophysical testing (during and after construction).

The applicability of these methods are discussed as follows:

Laboratory Testing

Laboratory testing is a valuable basis 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.

Field Wet Grab Sampling

The concept is to obtain samples from the treated ground before the mix reaches such a strength that a sampler cannot be introduced easily or without causing significant sample disturbance. 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 to 12 mm. The finer material is then used to prepare cylinders for strength testing.

Although the evidence is incomplete, available data suggests that the wet grab strength is typically less than one-half the strength obtained from core samples.

Coring

Core drilling is often used to obtain test specimens for quality assurance. 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 during installation. Limited data suggests that core strenght is on the order of 1/2 to 2/3 lab strength.

Experience at the Central Artery project identified several key elements which promote good and representative core sampling. These include using experienced drillers and logging engineers; taking large diameter cores (greater than 76 mm 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. 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.*

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 to 1.0 percent of installed columns.

Available data suggests that the core unconfined compressive strength is greater than that of wet grab samples and 60 to 80 percent of laboratory unconfined compressive strength.

Most of the improved strengths/values in table 3 are based on core test results.

Other methods

For verification of low strength soil mix product such as lime and LCC, in place probe pullout testing, pressuremeter and CPT testing have been routinely used.

Figure 17 illustrates the set up for the pullout probe test on LCC which is applicable for LCC strengths of less than 200 kPa.

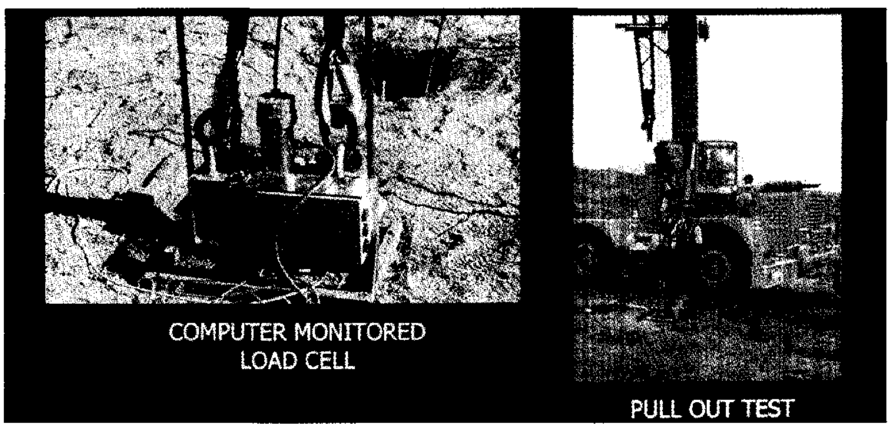

Figure 17. Pullout probe testing, LCC

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-destructure testing methods are conceptually promising but insufficient data to judge their accuracy and costs are presently available (1999).

CHAPTER 5

BIDDING METHODS, CONSTRUCTION SPECIFICATIONS AND CONSTRUCTION MONITORING

Since all DMM technologies are contractor driven and based on specific equipment and methods, performance type specifications appear to be appropriate. DMM projects to date have typically used performance specifications and the single highway LCC project was being 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 typical 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. 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 the 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. Suppliers 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 manufacturers 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 onsite 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 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 mm to 800 mm 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. 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 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 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 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 in width and 15 to 20 mm 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 which 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 and to confirm design strengths and 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 percent 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-initiation 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, and taking the undrained shear strength as 18% of the limit pressure. Alternately, complete columns can be recovered by driving a square 11 m long sampler, by vibratory methods, and recovering whole columns of stabilized soil for laboratory testing.)

(A verticality requirement may be added if LCC are used for excavation support.)

6. Measurement and Payment

(Lime cement columns may be paid on a L.M. basis as a total compensation for equipment, material and labor required for installation.)

b. Specifications for Deep Mixing Method (DMM)

Currently, DMM has a wide variety of uses. It is 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.

An amended version of DMM specifications used on projects at the Central Artery in Boston are included to form a preliminary basis for development of a site specific specification. 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 contractors 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

1.01 General

A. This Section specifies requirements for: furnishing and installing soil-cement by deep mixing to increase the compressive strength of subsurface soils over the plan area coverages and configurations, and depths, usually penetrating into *(project specific description of strata to be penetrated)*, shown on the Drawings, and alignment tolerances stated herein; and conducting a Quality Assurance-Quality Control Program to demonstrate that the installed soil-cement conforms to the requirements stated herein.

Deep soil-cement mixing shall be performed to mix *(describe in situ soils)* with cement grout to form a uniform mixture, blended thoroughly and evenly by controlled methods.

Installation of soil-cement by deep mixing auger methods shall create *(describe use)* soil-cement structures required to improve deep ground stability *(describe use)*.

The Quality Assurance-Quality Control Program shall confirm that the installed soil-cement achieves the required compressive strength and ensure that it is installed in the design configuration and within the specified limits.

B. Definitions:

1. Pre-Production (PPC) Program - Field test program undertaken by the Contractor as described in Section *(fill in as required)*.

2. Testing Laboratory - *(Fill in as required)* responsible for forming, curing, preserving, and transporting samples; performing laboratory testing; and reporting laboratory test results.

3. Soil-cement - Uniform mixture of cement grout and in-situ soils.

4. Soil-cement Element - A column or multiple columns of soil-cement formed vertically by injection of cement grout and, controlled mixing and blending with in-situ soils which result in a uniform mixture of cement grout and the in-situ soils in a single penetration.

5. Soil-cement Structure - A number of soil-cement elements that are continuously interconnected by required overlapping, which may be in the form of a wall or a mass.

6. Vertical Alignment Profile - A graphical or tabular data presentation of the actual alignment of a soil-cement element compared to vertical.

7. Spoil Return - All materials including, but not limited to liquids, semi-solids and solids which are discharged above ground surface or mudline, as a result of soil-cement mixing.

8. Obstructions - Objects or materials occurring at or below ground surface which prevents penetration of the auger to the required depth. Obstructions include, but are not limited to, concrete, bricks, stone blocks, wood piles, metal, abandoned foundations, and utilities and other items.

   Naturally occurring materials such as: cobbles, boulders, dense, well-bonded or other competent in-situ soils will not be considered as obstructions.

C. Equipment to perform deep soil-cement mixing shall work both on existing ground surfaces as well as surfaces of man-made fills placed over stabilized soils. Soil-cement mixing equipment shall be capable of advancing through previously installed and cured soil-cement.

D. Deep soil-cement mixing shall be performed utilizing the "Initial Production Parameters" established from the accepted PPC Program.

E. A Quality Assurance-Quality Control Program for the soil-cement shall be implemented during the course of the Work to confirm that the installed soil-cement achieves the required compressive strengths; unit weight and plan area configurations and coverages over the depths and limits shown on the Drawings, and alignment tolerances stated herein.

F. Related work specified elsewhere:

   (Fill in appropriate contract references)

1.02 Performance requirements for deep mixing

A. Stabilize subsurface soils by deep soil-cement mixing to provide *(describe)* and other required configurations of redrilling, repenetration, overlapping and interconnected soil-cement elements using patterns and installation sequence and schedule shown on the drawings. Acceptance of deep soil-cement shall require that the installed soil-cement elements be installed to the plan area configurations and coverages over the depths and limits shown on the Drawings, to the alignment tolerances, to the required compressive strength, and unit weights uniformity stated.

1.03 Qualifications

A. The firm(s) performing deep soil-cement mixing shall provide on-site, through all working hours Project Engineers throughout the full duration of the Work of this Section, experienced in work comparable to that described herein, and having at least five full years experience as full time responsible Project Engineers, within the past ten years, in deep soil-cement mixing. Experience as full time Project Engineer within the last ten years shall include at least two projects each of which had installations of at least 7,500 cubic meters of soil-cement to depths exceeding *(fill in as required)* meters and which required mixing in cohesive soils with overlapping soil-cement elements to create soil-cement wall structures similar to those required on this project.

B. Field Superintendents shall be required continuously on site, each shift of operation. The Field Superintendents shall each have at least three years accumulated experience with soil-cement techniques similar to that required for the Work specified herein; including at least two projects, one of which within the past five years requiring deep mixing in cohesive soils similar to that required on this project.

C. Soil-cement mixing Rig Operators shall have accumulated a minimum 100 hours of operating time on the specific soil-cement mixing rig under the direct supervision of the Field Superintendent and equipment manufacturer representative.

D. Grout mixing plant operators shall have a minimum 3 years experience in operating computer based cement grout mixing batch plants of comparable complexity to the equipment proposed for the Work of this Section.

E. *(The Contractor's personnel responsible for survey layout, lines and grades, shall be Registered Land Surveyor or a Registered Professional Civil Engineer.)*

(Submittals shall be stamped by a Professional Civil Engineer except for survey data which shall be stamped by a Professional Civil Engineer or a Registered Land Surveyor.)

F. The independent Testing Laboratory shall have at least five years experience as a materials testing laboratory including the performance of testing comparable to that required herein. The person in charge of the Work for the Testing Laboratory shall be a Professional Civil Engineer, registered in *(describe)*. The Testing Laboratory's supervisor and each field representative taking samples shall each have at least five years of experience in taking concrete samples in the field and performing compressive strength tests in accordance with AASHTO (or ASTM) requirements.

1.04 Submittals

A. At least four weeks prior to commencement of any mobilization of deep soil-cement mixing equipment for production mixing, the Contractor shall submit the following to the Engineer:

1. Names and qualifications of the soil-cement mixing personnel and surveyors, including project experience, resumes and other documentation that demonstrate the qualifications of the Project Engineers, each Field Superintendent, rig and batchplant operators for the deep soil-cement mixing.

2. A list of Contractors, responsible engineers, project descriptions and personnel responsibilities from soil-cement mixing projects completed by the soil-cement mixing Project Engineers and Field Superintendents during the past ten years. Contractor's names, addresses and telephone numbers shall be included for these projects representing the individuals comparable experience. The projects listed to demonstrate personnel qualifications shall have employed equipment using similar auger configuration(s) as proposed for the work of this Contract.

3. Submit data on equipment to be used for the deep soil-cement mixing, proportioning, pumping, injecting and mixing soil-cement as wells as all other ancillary equipment, including equipment capable of remixing non-conforming soil-cement.

4. Spoil containment *(sheet piling or other)* structures and methods to be used to prevent the migration or leakage of spoil return, disturbed in-situ soils or other spoil material beyond the immediate limits of soil-cement mixing operations. The Contractor shall demonstrate that the containment structure is stable under loads applied by soil-cement, water, in-situ soils, overlying fill materials, construction equipment, other surcharge loads and loads applied by the subsequent deep mixing operations. Include also details and methods to be used to collect and dispose of the spoil return and other spoil materials.

5. Sequence and time schedule of all operations including plan location and sequence of all deep soil-cement mixing. The Contractor shall submit a soil-cement element Layout Plan based on the patterns shown on the Drawings to achieve the required plan area configurations and coverages and necessary overlaps and auger re-penetrations over the depths and limits shown on the Drawings. Plan locations of all

proposed soil-cement mixing shall be shown on Layout Plans of suitable scale to clearly show the details of the layout. Soil-cement elements on the Layout Plan shall be numbered and dimensioned.

6. Cement grout mix design including: cement type, cement source, cement compound composition, water-cement ratio by weight and other pertinent details. Limit water-cement ratio to *(1:1 maximum)*, unless otherwise demonstrated in PPC.

7. Soil-cement mix design and procedures for production mixing shall be based upon the experience gained the PPC Program including: estimated in-situ 28-day *(or 56-day)* compressive strength of the soil-cement, cement-grout injection pressure and rates, mixing rotational speeds, penetration and withdrawal rates of the mixing tools, and mixing times at bottom of the soil-cement element when there is no vertical movement of the mixing tools, and complete description of all mixing operations.

8. Description of Quality Assurance-Quality Control Plan for deep soil-cement mixing including, but not limited to, the following:

   a. A detailed description of the Quality Assurance-Quality Control Program to be undertaken each day during soil-cement mixing to confirm that the installed soil-cement conforms to the required compressive strengths and unit weights specified, the plan area coverages over the required depths and limits, and required horizontal and vertical alignments.

   b. Details of procedures to obtain soil-cement samples, catalog cuts of the soil-cement sampling device and curing boxes.

   c. Measures to be implemented each day during soil-cement mixing to continuously monitor, modify and control: water-cement ratios; cement-grout injection pressures and quantities; mixing rotational speeds; penetration and withdrawal rates of the mixing equipment; horizontal and vertical alignments of the soil-cement elements; and other related aspects of the soil-cement mixing process.

   d. Example formats of Daily Production Reports conforming to the requirements stated herein.

9. Proposed details and formats of all required tabular and graphical data presentations to be submitted to the Engineer during the course of the Work.

B. Submittals shall be stamped by a Professional Civil Engineer registered in the State of *(fill in)*.

C. Within two business days after the completion of each soil-cement element, submit the following:

1. Deviations of the center coordinates from the layout plan to the nearest 75 mm at the top of the element.

2. Vertical alignment profiles shall be submitted in accordance with the frequencies specified to the nearest 13 mm over the measurement length along axes parallel and perpendicular to the line of longitudinal progression; the elevation to the nearest 30 mm of the top and bottom of the element.

D. Within one business day after the end of a work shift, submit Daily Production Reports for the work shift to the Engineer. Daily Production Reports shall be filled out, checked for correctness, and signed by the Deep Soil Mixing Firm's field superintendent and the Contractor's field superintendent at the end of every work shift. The reports shall contain, but not be limited, to the following information:

1. Day, month, year, time of work, shift, beginning and end; names of each superintendent in-charge of the Work for both the soil-cement mixing firm and the Contractor; a list of all workers' names associated with each soil-cement mixing machine; and a summary of equipment used during the shift.

2. The location and "neat" limits as shown on the Contract Drawings of each completed soil-cement element installed during the work shift and all soil-cement elements completed to-date, on a plan of suitable scale to clearly detail the locations of the elements.

3. Time of day of beginning and completion of each soil-cement element installed during the work shift.

4. Water-cement ratios, cement type, brand, and cement grout injection pressures and rates, mixing rotational speeds, penetration and withdrawal rates of the mixing equipment, batch plant production records, and installation sequence for every soil-cement element.

5. Other pertinent observations including, but not limited to; spoil returns, cement grout escapes, ground settlement and/or heave, collapses of the soil-cement element, advancement rates of the mixing equipment, and any unusual behavior of any equipment during the soil-cement mixing process and other noteworthy events. In the event of any Contractor claim, the Daily Production Reports shall be the primary documents to substantiate the reasons and basis for the claim.

6. Date, time, plan location, sample designation and elevation, and other details of soil-cement sampling.

7. Summary of any downtime or unproductive time, including start and end time, duration, and reason.

E. Within five weeks after completion of the deep soil-cement mixing, the Contractor shall submit survey data including:

1. A final Layout Plan of suitable scale showing the locations of each soil-cement element at the top of the element.

2. A tabular summary of layout center coordinates modified to reflect any measured deviations of each soil-cement element at the top of the element, the elevation of the top and bottom of each element.

3. A compilation of all vertical alignment profiles.

1.05 Quality Assurance-Quality Control Program

A. The Contractor shall implement a Quality Assurance-Quality Control Program to verify that the installed soil-cement elements conform to the requirements stated herein.

B. The Contractor shall obtain soil-cement samples, including fluid and core samples, and provide them to the Owner. The Owner will form preserve, cure, transport, and test the samples and report the test results. The Contractor shall, coordinate sampling activities, and the Quality Assurance Quality Control Program with the Owner and its Testing Laboratory. The Contractor shall supply incidental items, access, inside storage space, curing boxes and electrical power to the curing boxes. The Contractor will supply molds for use in forming the samples.

C. Wet Grab Soil-Cement Samples

1. A minimum of one in situ sampling round shall be performed at a frequency of once per day at locations selected by the Owner. The sampling round shall be obtained at the same element which shall consist of a non-cured soil-cement sample from three depths selected by the Owner. The Contractor shall at the direction of the Owner obtain up to 20 additional wet grab sample test rounds, at no additional cost to the Owner.

2. Separate soil-cement samples shall be retrieved within 60 minutes of the completion of the soil-cement column. The device used to retrieve the wet grab soil-cement samples shall be capable of obtaining a discrete fluid sample of soil-cement at a depth determined by the Engineer and shall be capable of accepting particles not thoroughly mixed that are up to 150 mm in dimension. The sampler shall be lowered empty, air only, to the required depth in the soil-cement element and then opened. Once filled with the soil-cement the sampler shall be closed to exclude entry or loss of soil-cement and shall be expeditiously raised to ground surface.

3. Each retrieved soil-cement sample shall be of sufficient volume to produce a minimum of four full cylinders, 150 mm diameter by 300 mm. Separate and retain all soil-cement retrieved from each depth, and immediately provide each to the *(fill in as appropriate)*. The *(fill in as appropriate)* will then cut all retrieved particles of soil larger than 25 mm into smaller pieces that will pass a 25 mm sieve, and then immediately form the four cylinders with material passing through a 25 mm sieve (including particles of soil that were cut up).

4. Soil-cement samples shall be formed, cured and preserved in accordance with AASTHO T 23 and protected from freezing and extreme weather conditions which could have deleterious effect, at all times.

5. The Contractor may obtain additional samples and perform additional testing for his own information, at no additional cost to the Department.

6. If the Contractor cannot obtain all of the required wet grab samples of the soil-cement, in the designated soil-cement element, the Contractor shall obtain a full suite of wet grab samples from the next soil-cement element installed by that rig. Continue taking wet grab samples in subsequent soil-cement elements until a full suite is obtained.

7. Use one cylinder from each sampling depth to determine the cement factor based on the cement content determined in accordance with AASHTO Specification T 144 (or ASTM D 806). Test the remaining cylinders for unconfined compressive strength in accordance with AASHTO Specification T 208 (or ASTM D 2166). Unless directed otherwise by the Engineer, test one cylinder at 7 days, one cylinder at 14 days and two cylinders at 28 days. Submit the laboratory test results to the Engineer in accordance with this specification.

D. Core samples:

1. Core samples shall be taken by the Contractor, for the purpose of obtaining and testing in-situ samples to evaluate compressive strength, unit weight, vertical alignment, and composition of the soil-cement. Coring of soil-cement shall be performed in accordance with AASTHO T 225 and the requirements stated herein.

2. Continuous core samples shall be obtained at up to *(fill in number)* separate locations selected by the Engineer and over the entire depth of the soil-cement. The samples shall be obtained using a PQ-size triple tube core barrel with a side discharge.

3. Immediately after retrieving the soil-cement core samples from a specific boring, wrap preserve and submit to the Engineer seven core samples per boring selected by the Engineer for subsequent evaluation and testing. Grout the borehole after coring.

4. Remaining core samples shall be boxed, stored, preserved and delivered. Core samples shall be protected from freezing and extreme weather conditions at all times.

5. If recovered core samples from any boring provide less than 90% recovery, or less than 50% RQD, or fewer than two intact cores of length more than eight inches, in each core run, the Engineer may direct the Contractor to drill up to 2 additional borings and recover additional core samples for testing. If the samples from either of these additional borings do not provide required recovery, this process shall be repeated until coring provides required samples. These additional borings and core sampling shall be done at no cost to the Owner.

E. Vertical Alignment Profiles:

1. The Contractor shall obtain vertical alignment profiles for one element per day.

2. The Contractor shall advise the Engineer within one hour after measuring the vertical alignment, of any non-compliance with tolerance requirements.

3. The Contractor shall, at the direction of the Engineer, obtain up to *(fill in number)* additional vertical alignment profiles, at no additional cost to the Owner.

1.06 Job Conditions

A. Subsurface strata may contain *(rubble, ballast fill, rip rap stone, concrete, timber cribbing, stone blocks, wood piles and sheet piling, abandoned foundations, utilities)* and other materials obstructing soil-cement mixing equipment. Known obstructions and areas of known obstructions are indicated in the Contract Documents.

B. The Contractor shall take all precautions necessary to prevent damage to any existing structure, including settlement, loss of ground, shifting, or heaving of ground that could occur due to soil-cement mixing in the vicinity of existing structures, roadways, railroads, bridges, seawalls or utilities.

C. Throughout execution of the Work, the Contractor shall take all precautions and measures necessary to safely move, position, and operate soil-cement mixing equipment, support equipment and personnel around the site.

D. Other site conditions include, but are not limited to, the following:

(Describe other important site conditions, adjoining structures and roadways, required coordination with other construction operations)

2.01 Cement Grout

A. Cement shall conform to AASHTO M85, Type II, and shall be as specified in the submitted cement grout mix design accepted by the Engineer. Slag cement or fly ash shall not be allowed without the acceptance of the Engineer and submission by the Contractor of test data which confirm no deleterious impact to the soil-cement.

1. 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. 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 and in amounts that may be deleterious to the grouting operations shall not be used.

3. All cement shall be homogeneous in composition and properties, and shall be manufactured using the same methods. Secure at least two sources of cement supply and provide documentation of the composition of the cement from each supplier. Throughout the course of the work, obtain cement from one plant and one supplier, retaining second supplier as back-up source.

4. Tricalcium aluminate content shall not exceed 8 percent.

B. Water used in soil-cement and cement grout mixing and all other applications shall be potable, clean, of neutral pH and free from sewage, oil, acid, alkali, salts, organic materials and other contamination.

C. Cement grout shall be a stable homogeneous mixture of cement and water. The ratios of the components shall be proposed by the Contractor, confirmed during the PPC Program and reviewed by the Engineer. Cement grout composition shall not change throughout the soil-cement mixing unless requested in writing by the Contractor and accepted in writing by the Engineer.

2.02 Soil Cement

A. Soil-cement shall be a stable, uniform mixture of cement grout and the in-situ soils. The properties listed below shall be verified throughout the course of the Work in accordance with the Contractor's Quality Assurance-Quality Control Program.

B. The ratios of various soil-cement components shall be proposed by the Contractor or as accepted during the PPC Program subject to review by the Engineer. The Contractor shall adjust the mix design throughout the course of the Work in order to achieve the required compressive strengths and total unit weights. The Contractor shall submit changes in the mix design or cement factor and obtain the Engineer's acceptance prior to implementing these changes. The Contractor shall also adjust the mix design when directed by the Engineer.

C. Soil-cement obtained from wet grab samples shall conform to the following minimum compressive strength requirements. Unconfined compressive strength testing shall be performed in accordance with ASTM D2166:

1. Soil-cement shall achieve a 56-day *(or 28-day)* unconfined compressive strength of f'c equal to *(fill in)*.

2. The average unconfined compressive strength within each soil cement element shall be *(fill in)*.

D. The total unit weight of soil-cement samples shall be measured and shall be at least *(fill in)* or as determined and accepted by the Owner. For each test suite the average total unit weight of all soil-cement samples within the round will be calculated. If the average total weight from any two consecutive test rounds is less than *(fill in)* the Contractor shall adjust its mix as necessary to achieve the required unit weight.

E. Conformance with soil-cement uniformity criteria will be determined by the Engineer by evaluation of core samples. The soil-cement shall contain soil fragments with a maximum dimension not to exceed 1/4 of the diameter of the auger or 300 mm whichever is smaller. In addition, seventy percent of the depth cored shall have a minimum core sample unconfined compressive strength of *(fill in)*.

Note that strength evaluations for quality control based on core samples of a given age (28 or 56 days) may be specified in lieu of strength based on wet grab sampling. A maximum unconfined compressive strength limit may be specified if the soil mix volumes are subsequently excavated.

2.03 Minimum Equipment Requirements

A. Equipment shall be maintained so as to ensure safe, continuous, and efficient production during soil-cement mixing and other related operations.

B. Equipment shall have devices to permit accurate and continuous monitoring and control of: water-cement ratios, cement-grout injection pressures and quantities, mixing rotational speeds, advancement and withdrawal rates of the mixing tools, and other operations required to install and mix the soil-cement.

C. The soil-cement mixing machines shall be sufficient size, capacity, torque, and capable of performing deep mixing to the required depths shown on the Drawings. The soil-cement mixing machine shall be capable of advancing and withdrawing the mixing tools while simultaneously injecting cement-grout and mixing in-situ soils.

D. Deep soil-cement mixing equipment shall use single or multiple-shaft mixing equipment with multiple *auger* centers configured in one straight line. Shafts shall uniformly inject cement grout at the bottom of the assembly. Mixing equipment shall be capable of advancing through previously installed and hardened soil-cement. Continuous flight augers longer than 1.5 m are not allowed. Auger flights and mixing paddles on a shaft shall extend to the full diameter of the element being formed, and shall have discontinuous lengths and be spaced to overlap with paddles of adjacent shafts in order to thoroughly break up the in-situ soils and blend them with injected cement grout to form a homogeneous mixture. If used to breakup and blend in-situ soil and cement grout, high pressure jets shall not be directed radially to extend beyond the perimeter of the auger flights. Air shall not be injected into the in-situ soils.

E. *(The soil-cement mixing machine shall be equipped with inclinometer-type instrumentation to measure, record, and produce vertical alignment profiles over the required length of the soil-cement element along axes both parallel and perpendicular to the line of longitudinal progression. The instrumentation on the soil-cement mixing machine shall be capable of measuring the deviation from vertical to the nearest 13 mm horizontal in 30 m vertical.)* Include if required.

F. The cement grout batching plant shall include all storage silos, weather protection, sheds, scales, pumps, mixers, valves, gauges and regulating devices required to continuously measure and mix cement grout.

G. Confirm performance by undertaking a deep mixing demonstration program prior to commencing production soil mixing. During this demonstration program the contract requirements for vertical alignment, wet grab sampling, and the requirements of Article 2.02, shall be performed all at no additional cost to the Owner.

3.01 Construction Methods - General

A. The Contractor shall be responsible for furnishing the labor, equipment, and materials necessary to conduct all soil-cement deep mixing operations.

B. The Contractor shall perform for all survey layout, and utility clearance as affecting the soil-cement mixing and coordination with all local, state, and federal agencies having jurisdiction.

C. The Contractor shall mobilize and maintain a sufficient number of soil-cement mixing machines, materials, cement grout batching plants, and crews to complete the Work in accordance with project milestones.

D. Within 14 Days after receiving the Engineer's notification of PPC acceptance and authorization to proceed with deep soil-cement mixing, mobilize to the site the equipment, labor and materials required, and begin substantive deep soil-cement mixing activities.

E. The Contractor shall coordinate soil-cement mixing operations with all other aspects of the Work.

3.02 Installation of Soil-Cement

A. Soil-cement mixing shall be performed to construct continuous soil-cement structures in wall and other arrangements to achieve the required overlap continuity for the plan area configurations and coverage shown on the Drawings, using the installation sequence and schedules there shown, and with soil-cement elements conforming to the alignment tolerances and required compressive strength and unit weights stated herein.

B. Overlap and redrilling of adjacent Soil-Cement elements along the line of longitudinal progression shall be for the distances indicated on Contract Drawings.

C. Deep soil-cement shall be installed with the same make and model of; mixing machinery, cement grout mixing and pumping equipment, and the same materials and procedures implemented by the Contractor and accepted by the Engineer in the PPC Program as "Initial Production Parameters." The Contractor shall adjust the mix design, as necessary, throughout the course of the work in order to achieve the required compressive strength and total unit weight. The Contractor shall submit changes in the mix design to the Engineer for the review and acceptance prior to implementing the changes.

D. Soil-cement elements shall penetrate *(describe strata to be penetrated or elevation to be reached)*.

Upon reaching the bottom of the soil-cement element, the mixing equipment shall be operated at sufficient speed and duration to clean and mix all loose, soft and otherwise unmixed soil prior to final grouting and withdrawal of the mixing tools.

E. During soil-cement mixing, grout shall be introduced only by injecting cement grout through the bottom of the operating mixing equipment. Grout shall be introduced during the initial penetration of the *augers*, or during subsequent downstrokes of the *augers*, for the entire depth of the elements.

F. *(In the event geotechnical instrumentation specified indicates Contract Threshold and/or Limiting Values are reached and such movement is related in any way to the soil-cement mixing operations, the Contractor shall meet with the Engineer and implement the proposed action plan to restrain and control movements within allowable ranges. The Contractor shall stop soil-cement mixing operations if necessary in order to limit movements or implement the proposed action plan.)*

G. After final grouting of the soil-cement element, the Contractor shall obtain samples of in-situ soil-cement samples in accordance with the locations and frequencies required in the Quality Assurance-Quality Control Program.

H. During the course of all soil-cement mixing, no water, debris or spoil material shall be dumped or otherwise allowed to enter the soil-cement element.

I. Any soil cement element which exhibits partial or total instability, shall be backfilled with weak cement grout and be remixed full depth, at no additional cost to the Owner. Among other possibilities, signs of instability which could be observed in the field during the work could include excess flow of grout from the hole, settlement of ground, and squeezing in on the drill rods.

3.03 Horizontal and Vertical Alignment Tolerances

A. The maximum horizontal deviation of the as-installed center of any soil-cement element at the ground surface or mudline installation level shall not exceed 75 mm from the layout center coordinate, shown on the accepted Contractor's submittal.

B. The measured vertical alignment of soil-cement elements shall not deviate in any direction more than two percent from vertical of the measured length, or be inclined more than two percent from vertical anywhere along the measured length.

C. At the direction of the Engineer, any soil-cement elements which exceeds the allowable horizontal or vertical alignment tolerances shall be re-mixed or supplemented with one or more adjacent or overlapping elements, at no additional cost to the Owner.

3.04 Obstructions

A. *(Subsurface strata, may contain rubble, concrete, reinforced concrete slabs, metal, bricks, granite stone and blocks, wood piles, seawalls, depressed trackway structures, abandoned foundations, utilities and other materials that can obstruct soil-cement mixing.)* Prior to performing deep soil-cement mixing, the Contractor shall complete any required demolition work to remove all known obstructions. If the Contractor does not remove known obstructions during the demolition phase, the Contractor shall remove obstructions during the deep soil-cement mixing operations or install additional soil-cement to encapsulate by deep mixing or other methods the obstruction, as directed by the Engineer, at no additional cost to the Owner.

B. Where obstructions are encountered during deep soil-cement mixing, the Contractor shall remove the obstruction or install additional soil-cement to encapsulate the obstruction, at the direction of the Engineer. Each situation shall be resolved on a case by case basis. If such conditions are encountered, the Contractor shall notify the Engineer in writing, and provide all pertinent information relating to the nature, depth, plan location coordinates, expected extent of the obstruction, and proposed procedures to overcome the obstruction. Such construction to overcome an unknown obstruction shall only be performed with the written authorization of the Engineer, and shall be compensated as additional measured quantities of deep soil-cement as directed by the Engineer.

C. If difficult drilling is encountered due to the presence of naturally occurring cobbles, boulders or dense well-bonded soils or other characteristics of the naturally occurring soils, the Contractor may elect to remove the hindrances, install soil cement by an alternate soil-cement mixing pattern that avoids or encapsulates the hindrance but that achieves the required soil-cement structure continuity, subject to the approval of the Engineer and at no additional cost to the Owner. Such naturally occurring conditions shall not be the basis for additional, time, measurement or compensation.

3.05 Containment, Collection and Disposal of Spoil Return

A. At all times during and at completion of soil-cement mixing operations, the site shall be maintained cleared of all debris and water. Spoil return and other spoil material shall be piped or channeled to holding ponds, tanks, or other retention structures or facilities. The Contractor shall remove and dispose of all waste materials daily in accordance with the requirements of the Owner, the Department of Environmental Protection and all other agencies having jurisdiction.

B. All soil-cement collection, containment and disposal methods shall be thoroughly explained and shown on shop drawings in the Contractor's submittals to the Engineer prior to the start of deep soil-cement mixing. *(For over-water soil-cement mixing, spoil return and any disturbed soils shall be contained within the sheet pile containment and turbidity control barrier.)* The Contractor shall be responsible for and incorporate all sedimentation and turbidity control measures required by applicable federal, state and local regulations.

C. The Contractor shall take all necessary precautions and implement measures to prevent any spoil return, other spoil material or stockpiled materials from entering storm drain structures, drainage courses and other utility lines or from leaving the site via surface runoff. The Contractor shall prevent the migration of spoil return, spoil material or stockpiled materials into any surface water body, beyond the immediate limits of soil-cement mixing operations.

4.01 Method of Measurement

A. Installation of soil-cement will be measured per cubic meter, to the nearest cubic meter, within only the "neat" plan area of the proposed soil-cement shown on the Drawings or approved by the Engineer. The volume shall be determined by multiplying the "neat" area within this zone times the actual length of the soil-cement elements which achieve the required depth. Areas of overlapping and overdrilling will not be measured more than once.

(Soil cement for perimeter walls are usually paid on the basis of square meters of wall in full compensation for all equipment, materials and labor.)

1. No separate measurement will be made for additional quantities of soil-cement installed to overcome obstructions.

2. Quantities of soil-cement installed by the Contractor during remixing to achieve the performance requirements due to augmentation required to supplement non-conforming elements, or that are outside the limits of soil-cement mixing shown on the Drawings without the acceptance of the Engineer will not be measured for payment. *(No distinctions will be made between on-land and over-water soil-cement measurement quantities.)*

3. No separate measurement will be made for; the Contractor's Quality Assurance-Quality Control Program, the supplemental full depth test suites of wet grab samples, all of which shall be considered part of the Work of soil-cement mixing, except for core borings.

B. Core borings will be measured for payment under Section *(xx)*

C. Mobilization and demobilization will be measured for payment under Section *(xx)*

D. Transportation and disposal of soil mix spoil will be measured for payment under Section *(xx)*. *(This item may be considered incidental and not paid separately. This would encourage contractors to limit spoil by usage of best available methods and equipment.)*

4.02 Basis of Payment

A. Installation of soil-cement will be paid at the contract unit price per cubic meter and considered to include full compensation for furnishing all equipment, materials and labor required to install soil-cement in accordance with the specified strength requirements in accordance with the plan area configurations and coverages over the depths and limits shown on the Drawings and requirements herein, implementing the Contractor's Quality Assurance-Quality Control Program and making the supplemental wet grab samples and vertical alignment profiles.

4.03 Payment Items

Installation of Deep Soil-Cement	Cubic Meter
(Installation of soil cement perimeter walls	*Square Meter)*

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 which 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 contractors 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 which 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 DMM and LCC 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.

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 grouped process control in three broad categories based on the degree of sophistication as follows:[1]

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	UCS test	Random sampling

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.

Level 2

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 which include the numbers 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 3

The highest level of computer control and display is provided. It features a microprocessor which senses, at short time intervals, rpm, penetration rate, torque, thrust, slurry density, pressure, 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.

Level 3 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 basis of construction monitoring.

CHAPTER 6

COST DATA

Deep soil mixing cost data is available from over a dozen projects completed in the last decade in the United States. By contrast only one lime-cement project has been implemented under a design-build contract and therefore cost information could not 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 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 $100 to $150 per cubic meter of volume treated is a reasonable range for large complex projects. Smaller projects using single auger methods should be budgeted at $60 per cubic meter 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 walers, tieback anchors, etc. for wall applications.

6.2 BUDGET COSTS FOR LIME-CEMENT COLUMNS

It is reported that costs in Scandinavia are in the range of $10 to $20 per linear meter 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 is $30 L.M. On average this would suggest a cost of $60 per cubic meter.

Costs are material sensitive with approximately 40 percent of the total cost attributed to reagent materials. For maximum construction efficiency, the columns should be in the range of 10 to 20 m in length.

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 DMM, FOR THIRD TUNNEL PROJECT, BOSTON CENTRAL ARTERY.[8]

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 DMM technology is being 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 18. The excavation was 1,100 m long, with the depth ranging from 12 to 26 m. The majority of the cut sections were supported by a soil mix wall to provide a ground water cut-off and a structural excavation support. Additional lateral support was provided by 2,700 prestressed soil anchors. An approximately 915 linear meter long soil mix wall with a total wall area of 37,160 m^2 was designed and constructed. The soil mixed wall was constructed using 0.9 m 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.

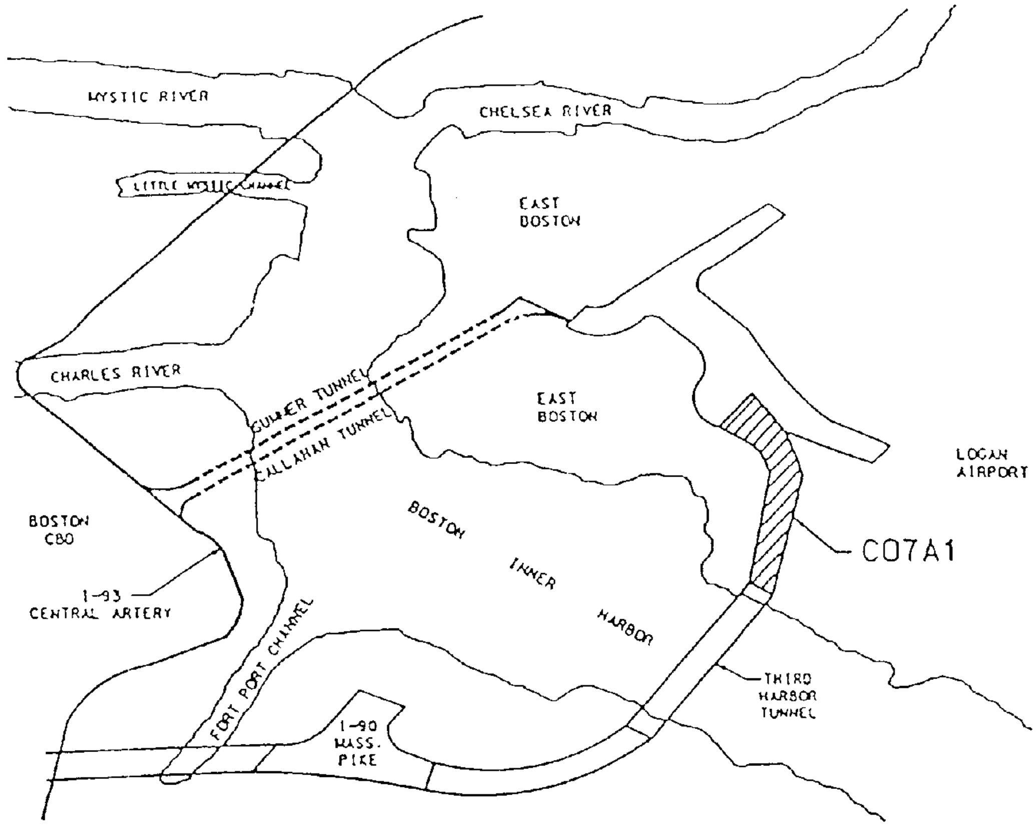

Figure 18. Site plan for deep mix wall - Boston Central Artery.

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 which 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 zones A and B.

In all areas, the ground elevation was approximately 34 m and the design ground water table was at about 33 m.

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 ground water 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 which accommodated the varying anchor angles and horizontal spacings, 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 is 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 is assumed that the bond to the steel piles is 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 operator of the automated batch plant to begin the pumping of the cement grout to the augers.

The project soil profile generally consisted of 6 to 7 m 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. 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 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 percent of the total soil treated. In marine clays, the excess quantities were as high as 50 percent. 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 dump site. 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 19).

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 cylinders were performed at 7 and 28 day intervals. Once the wall was cured in situ (times vary 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 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.

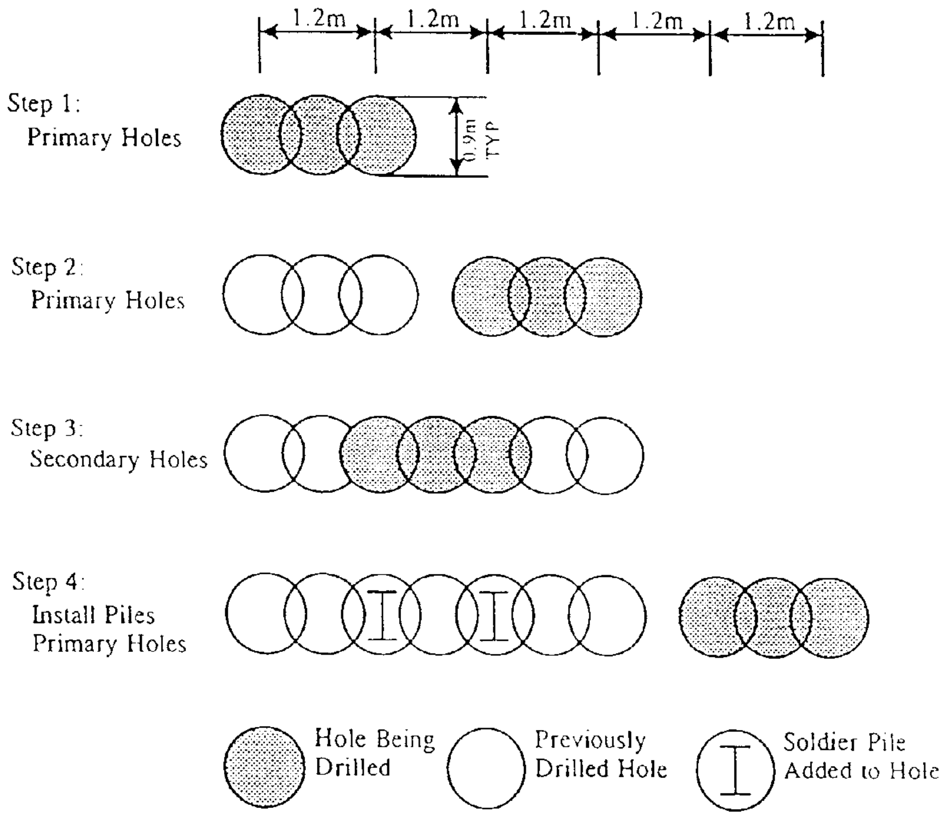

Figure 19. Soil mixed wall panel installation sequence.

Over 50 percent 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 percent were subjected to a more rigorous cyclic performance test.

Figure 20 shows the nearly completed construction.

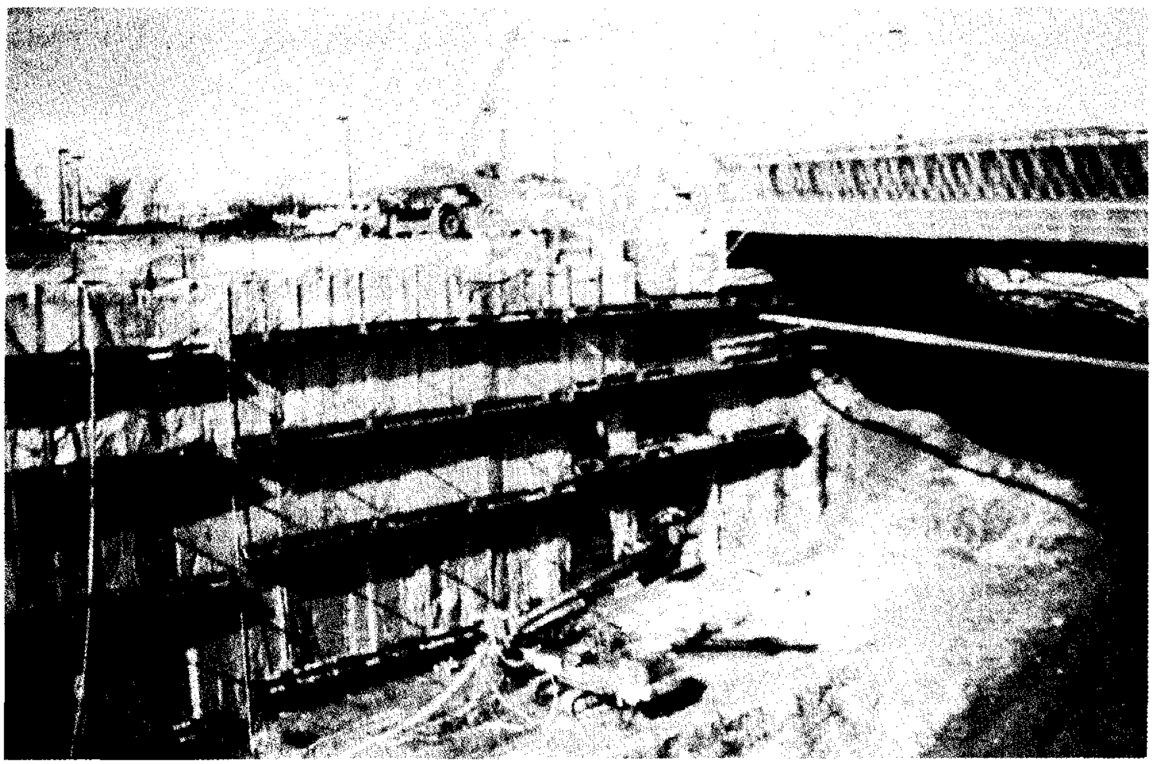

Figure 20. Nearly completed soil mix wall.

Additional Ground Treatment

In Zone C, a section of approximately 152 m which was underlain by marine clay to a depth of greater than 26 m experienced movement during construction. During the excavation to 13 m for the installation of the third row of anchors, the east wall moved 50 to 76 mm. A section of approximately 61 m continued to move 150 to 230 mm 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 21. 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 16).

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.

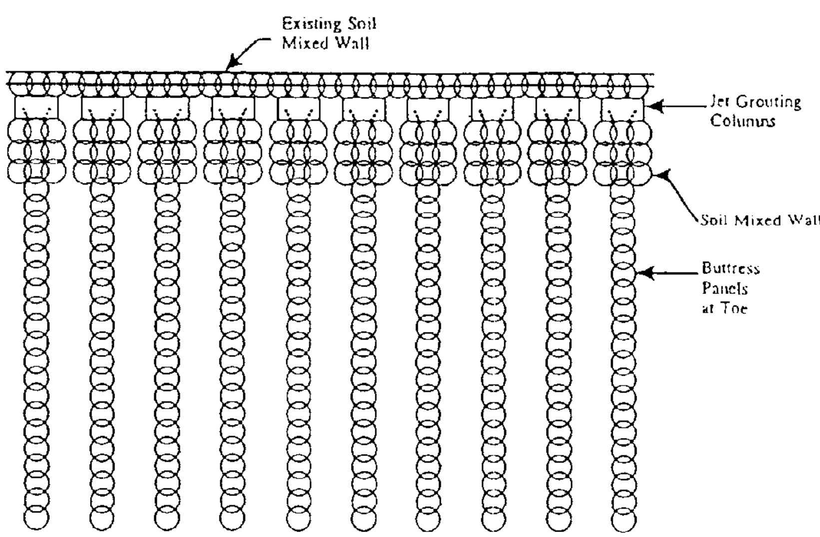

Figure 21. Soil mixed wall buttress for toe installation during excavation.

Summary and Conclusions

The use of an anchored soil mixed excavation support system on this project has 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 accommodates 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 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 lime-cement columns in the United States at I-15, Salt Lake City.[5]

LCC stabilization was used to support a mechanically stabilized earth wall (MSE wall) SS-01 located along west bound I-80 between the bridge crossing the UTA Light Rail corridor (200 West) and 300 West. The MSE wall heights ranged between 10m and 12.5m. A surcharge of as much as 3m 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 can 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 long and 800mm and 600mm 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 12m of clay with overconsolidation ratios (OCR) of 1.2 to 1.5 below a 3m crust. Water content ranged from 30 to 45 percent and P.I.'s on the order of 20. Below this upper clay a 1 to 2m dense sand layer was encountered which is typically underlain by interbedded sand and clay layers to great depth. Ground water was encountered at about 3m below grade and artesian pressure from a depth of 18m were observed.

Laboratory Testing

Laboratory unconfined compression, on laboratory prepared samples testing 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 on figure 22.

Based on the results of the testing and the design strength required, a mixture of 85 percent cement and 15 percent lime, 125 kg/m^3 was selected for use at I-15.

Design

The MSE wall is supported by panels of overlapping 800mm columns approximately 22m long for the full width of the wall (approximately 0.7H) plus 2m in front and 2m behind the wall. The panels are spaced 2m center to center (2 ½ diameters and have two cross-connections around the centroid of loading of the MSE wall.

The retained fill behind the MSE wall is supported on 600mm columns installed in a rectangular pattern between 1m and 1.2m 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 were installed beneath the back slope of the fill.

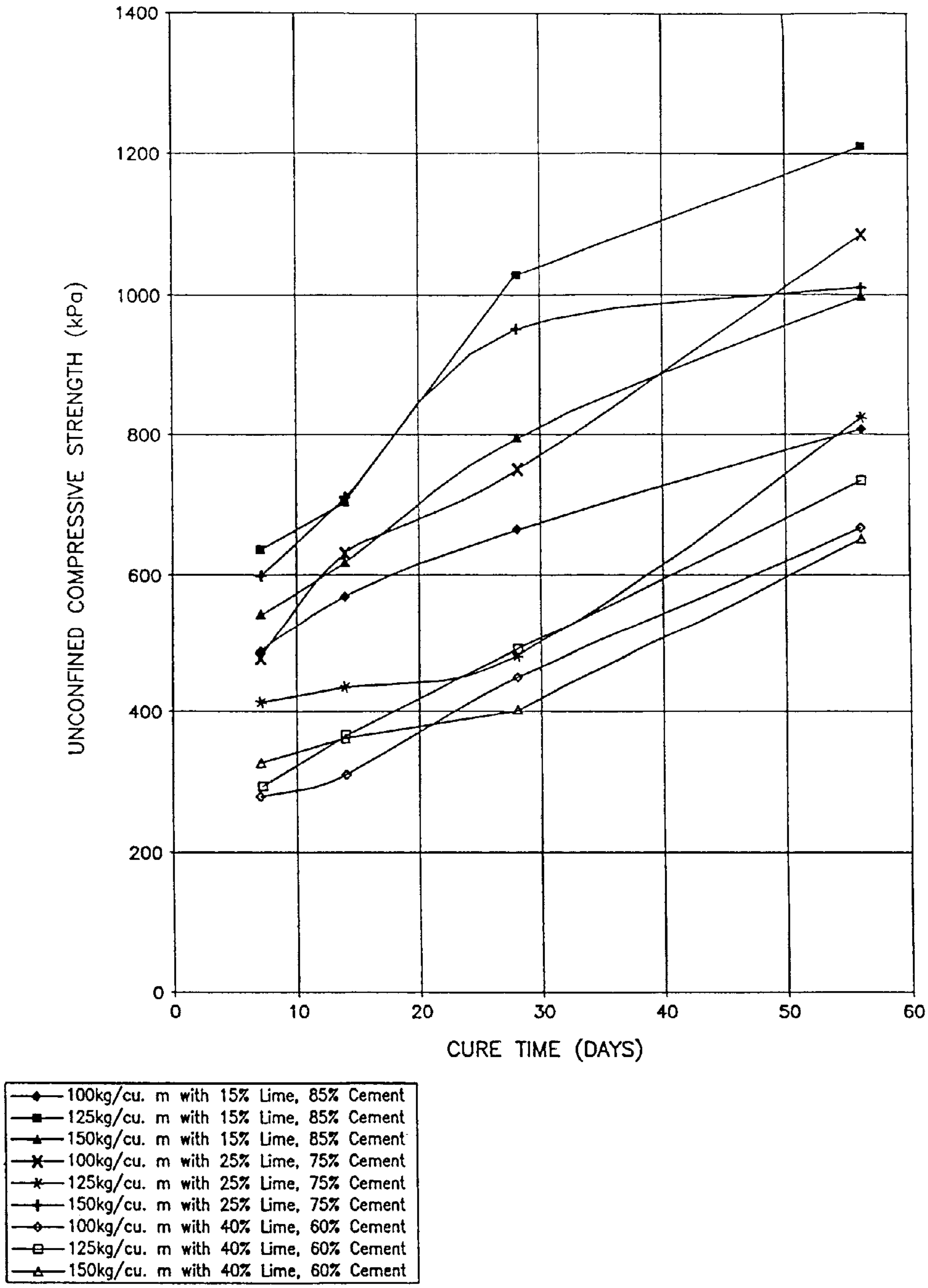

Figure 22. Unconfined compressive strength vs. time.

The 800 mm diameter columns were designed for a compressive strength of 600 kPa (undrained shear strength of 300 kPa). The 600 mm diameter columns, where better mixing was expected because of their smaller diameter, were designed for a compressive strength of 800 kPa. The undrained shear strength of the untreated Lake Bonneville clay at a point just below the desiccated upper crust is about 45 kPa.

Performance Data

Only limited data have been published to date on this project. Figure 23 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 100mm. Total settlement of the stabilized ground under the applied wall loads were estimated during design as between about 200mm and 300mm. Projections of the settlements measured to date indicate that total settlements are likely to be less than 150mm. The settlement of the wall if wick drains were to have been used to permit its construction was estimated as about 1300mm.

The data in figure 24 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 kPa and 250 kPa (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 was found transmitted to the soil and about 400 kPa transmitted to the columns. For the approximately 40 percent 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 and about 500 kPa. The stress on the soil between the panels is predicted by theory to be between about 50 kPa and 60 kPa. The agreement between the simple theory used for these calculations and the measurements appears excellent.

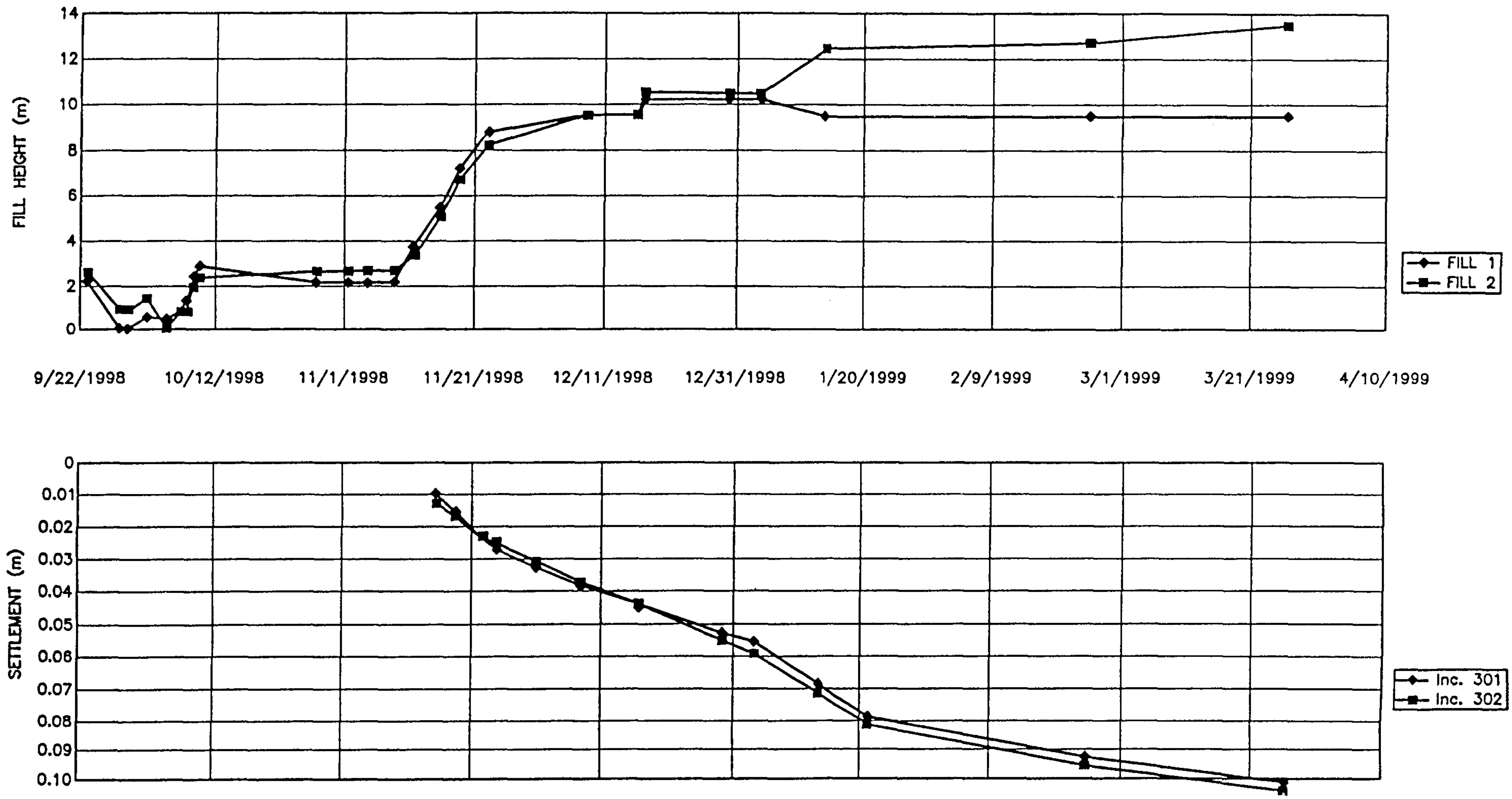

Figure 23. Fill height and maximum settlement vs. time.

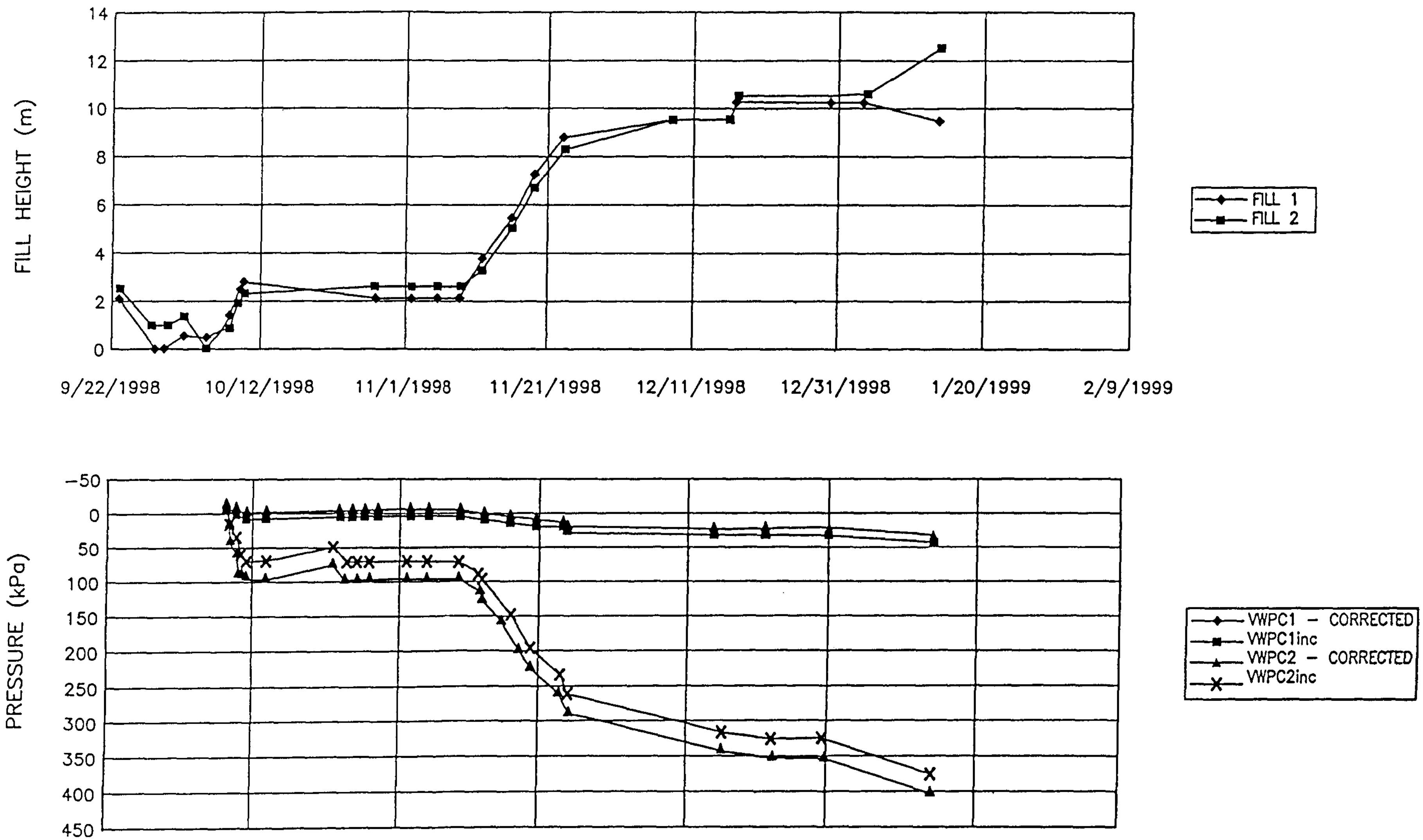

Figure 24. Fill height and stress on load cell vs. time.

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 have 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 12m to 15m 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 25 shows the equipment installing the lime-cement columns.

Figure 25. I-15, lime-cement column construction.

REFERENCES

1. Bruce, D. (1998). "An Introduction to the Deep Mixing Methods used in Geotechnical Applications, Verification and Properties of Treated Ground," *FHWA RD-99-205.*

2. Broms, B.B. (1993). "Lime Stabilization," *Ground Improvement*, M.P. Moseley, Ed. Blackie Academic and Professional, Boca Raton, FL. pp 65-99.

3. Terashi, M (1990). "Deep Mixing Method - Design and Practice and Soil Modification Recent Topics," *Technical publication of the Japanese Society on Soil Mechanics and Foundation Engineering*. Originally prepared for Japan - Indonesia Joint Symposium on Soft Soils in Construction and Deep Foundation, Jakarta, 1987. pp 25.

4. Hartlem, J. and Holm, G. (1995), "Deep Stabilization of Soft Soils with Lime - Cement Columns," *Proceedings from Bengt. B. Broms Symposium on Geotechnical Engineering*, Singapore, 1995. pp 173-179.

5. Esrig, M.I., MacKenna, P.E. (1999), "Lime Cement Column Ground Stabilization for I-15 in Salt Lake City," *34th Annual Symposium on Engineering Geology and Geotechnical Engineering*. Utah State University. April, 1999.

6. Yonekura, R., Terashi, M. Shibakazi, M. (Editors) (1996), "Grouting and Deep Mixing," *Proceedings of the IS-Tokyo 96/The Second International Conference in Ground Improvement Geosystems*, Tokyo, Balkema.

7. Lin, Q.K., Wong, H.I. (1999), "Use of Deep Cement Mixing to Reduce Settlement at Bridge Approaches," *ASCE Journal of Geotechnical and Geoenvironmental Engineering*. April, 1999. pp 309-320.

8. Ryan, C.R. and Jasperse, B.H. (1989). "Deep Soil Mixing at the Jackson Lake Dam," *Proceedings from ASCE Specialty Conference Foundation Engineering: Current Principles and Practices*. F.H. Kulhawy, Ed. ASCE, New York, NY. June, 1989. pp 354-367.

9. Yang, D.S. and Pearlman, S.L. (1995). "Boston Central Artery/Tunnel CO7 Project-Excavation Support and Ground Treatment."

10. Taki, O. and Yang, D.S. (1989). "Excavation Support and Groundwater Control Using Soil-Cement Mixing Walls for Subway Projects," *Proceedings from Rapid Excavation and Tunnelling Conference*. Society of Mining, Metallurgy, and Exploration, Inc. Littleton, CO. pp 156-175.

11. Taki, O. and Yang, S.D. (1991). "Soil Cement Mixed Wall Technique," *Proceedings from 1991 ASCE Special Conference*, Denver, CO.

12. Saitoh, S., Suzuki Y., Nishioka, S., Okumura, R. (1996), "Required Strength of Cement Improved Ground," *Proceedings of IS-Tokyo 96*, Balkena.

13. O'Rourke, T.D., McGinn, A.J., Dewsnap, J., Stewart H.E. (1997), "Performance of Excavations Stabilized by Deep Soil Mixing," *Cornell U. report to B/PB, Mass. DPW and FHWA,* Unpublished.

14. ______ (1997), "Deep Mixing" and "Lime-Cement Columns," *Ground Improvement, Ground Reinforcement, Ground Treatment. Developments 1987-1997.* ASCE Geotechnical Special Publication 69. V. Shaefer, Editor, Geo Institute of ASCE, New York.

15. Kivelo, M., Broms, B.B. (1997), "Undrained Shear Strength of Lime-Cement Columns," *Proceedings from XIV International Conference in Soil Mechanics and Foundation Engineering*. Hamburg, Balkena.

16. Ahnberg, H., Holm, G., Holmqvist, L., Ljungcrantz (1994), "Use of Different Additives in Deep Stabilization of Soil Soils," *Proceedings of 13th International Conference on SMFE*, New Delhi.

17. Hardiante, F.S. and Ericson, W.A. (1995). "Waste Clay Stabilization with Lime Columns," *Proceedings of Geoenvironment 2000--Vol. II.* Geotechnical Special Publication No. 46. Y.B. Acar and D.E. Daniel, Eds. ASCE, New York, NY. pp 1396-1407.

18. Barnes, B.L., Ericson, W.A. and Madrid, L.D. (1993). "In-Place Stabilization of Waste Phosphatic Clays Using Lime Columns," *Proceedings of the Third International Conference on Case Histories in Geotechnical Engineering*. University of Missouri, Rolla, MO. June 1-4, 1993. pp 1301-1304.

GROUTING

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CHAPTER 1

DESCRIPTION AND HISTORY

From the early days of simple slurry injections to today's sophisticated techniques, grouting has played an important role in the construction and upgrading of transportation facilities. New grouting technologies continue to be developed and existing technologies to be refined, while the range of applications continues to expand. This chapter discusses the history and development of grouting and provides an overview of the numerous geotechnical grouting techniques and their applications.

1.1 DESCRIPTION

Grouting comprises a variety of techniques which employ the injection of a range of materials into soil or rock formations, via boreholes, to alter the physical characteristics of the formation when the materials set. More specifically, grouting can be used to fill fissures and voids in rock, to fill voids between the ground and overlying structures, and to treat loose formations to enhance strength, density, permeability, and/or homogeneity. The type of grouting method used depends on such considerations as the project's specific requirements, the soil or rock type, and its amenability to different kinds of grout. An integral component of the program is a thorough geotechnical investigation to identify the site conditions and to logically guide the choice of the grouting method and its effectiveness.

1.2 FOCUS AND SCOPE OF MANUAL

Grouting technology continues to advance at a fast pace. With the development of newer techniques such as jet grouting and soil fracturing, and the progressive refinement of more traditional methods such as permeation and compaction grouting, today grouting offers a viable, engineered solution to a wide range of problems, including many in transportation facilities.

There is a vast body of published information on each of the types of grouting but much of it is in a format that renders it difficult to assimilate and implement by the engineering community at large. There is therefore a clear need for a working guide to grouting and its applications that will provide the user with a basis for decision making. This technical summary is designed to serve as that guide.

The focus and scope of this technical summary is to identify the types of problems that can be solved by grouting and to provide the user with sufficient information to make a preliminary technical and economic evaluation. Based on that evaluation, the potential for a grouting solution may be investigated further.

Chapter 1 provides an historic overview of grouting and introduces the various techniques available. Chapter 2 presents common aspects of grouting programs, Chapters 3 through 5 address each type of grouting individually. Design considerations, construction methods and cost data are discussed. Further references are included for in-depth subject research, and applications of each grouting technique are illustrated with project case histories.

1.3 GLOSSARY OF KEY TERMS

There is, as yet, no internationally adopted glossary of terms relating to grouting. Word meanings and interpretations vary from country to country. The following list represents some of the standard terminology used in the United States, and includes many of the definitions proposed by the American Society of Civil Engineers (ASCE).[1]

Additives – Additional grout components such as admixtures, bentonites, mineral additives, or pozzolans, such as pulverized fly ash, blast furnace slag, and condensed silica fume.

Admixtures – An added reagent which improves the grout in a specified manner through chemical or physical action. Examples of admixtures include accelerators, air-entraining agents, anti-freezing agents, dispersants, foam agents, plasticizers and super-plasticizers, retarders, water reducers, and underwater agents.

Aggregate – Loose, particulate materials, such as sand, gravel, pebbles, or crushed rock, added to a cementing agent.

Ballast – Material added to the cement grout to lower grout costs and improve some properties. (Also known as "filler").

Batch – The amount of grout mixed at one time.

Bentonite - Clay mineral with sodium montmorillonite as its main constituent. It is used as a viscosifier and a swelling agent.

Bingham fluid - A fluid with plastic viscosity, e.g., cement based grouts.

Blaine - The specific surface area of a substance, measured in cm^2/g or m^2/kg. Portland cements have a Blaine value of 3,000 to 5,000 cm^2/g. Microfine cements have a Blaine value of over 8000 cm^2/g and can reach 11,000 cm^2/g.

Bleeding - Water separating from grout under gravity head (decantation).

Bonding - Adhesion or the grip of cement to applied surfaces, i.e., interface strength.

Bulk density - The weight per unit volume of a material in its natural state.

Cement grout - A suspension mix of cement, water, air and sometimes pozzolan.

Chemical Grout - A material generally comprising a pure solution, or, in the case of sodium silicates, a natural colloidal solution, as used in permeation grouting operations. Distinct from *Particulate Grout* (below).

Colloidal - A state of suspension in a liquid medium in which extremely small particles (10^{-7} to 10^{-9} m) are suspended but not dissolved.

Colloidal mixer - A grout mixer that tries to emulate the colloidal state in a particulate grout.

Compaction Grout - Grout of low mobility and high internal friction, injected with less than 25 mm slump. Normally a soil-cement with sufficient silt sized component to provide plasticity, together with sufficient sand sized component to develop internal friction. The grout does not enter soil pores but remains in a homogeneous mass that gives controlled displacement to compact loose soils, and/or gives controlled displacement for lifting structures, and/or provides a controlled filling of large voids. Higher slumps may be used in void filling operations.

Darcy's law - The velocity of flow of a liquid through a porous medium because of a difference in pressure, is proportional to the pressure gradient in the direction of flow:

$V = Q/A = K \times dh/dL$

Where, V = velocity (m/s)
Q = flow rate (m^3/s)
A = cross-sectional area (m^2)
K = coefficient of permeability (m/s)
dh/dL = hydraulic (or pressure) gradient ($m/m = I$)

Emulsion - Colloidal particles dispersed and suspended in a fluid.

Fly Ash - The finely divided residue resulting from the combustion of ground or powdered coal which is transported from the fire box through the boiler by flue gases.

Gel - A semi-rigid colloidal dispersion of a solid in a fluid.

Gel time - The time required for a liquid material to form a gel under specified conditions of temperature.

Grout - A material injected into a soil or rock formation to change the physical characteristics of the formation after it has set or stiffened.

Groutability Ratio of Granular Formations - The ratio of 15 percent size of the formation particles to be grouted to the 85 percent size of the grout particles (suspension-type grout).

Grouting - The injection under pressure of a fluid that penetrates cavities, pores and/or fissures and then solidifies, making the formation more strong, compact, homogeneous, and/or watertight.

Hydration - The process of a cement or pozzolan reacting chemically with water.

Laminar flow - Fluid moving in layers with a difference in speed between the layers (center layers moving more quickly).

Microfine cement – A mixture of finely ground Portland or slag based cement often with mineral admixtures. Also known as Ultrafine cement.

Mortar – A cement-grout mixed with sand.

Newtonian fluid – In rheology, a fluid deforming for any applied stress.

OPC – Ordinary portland cement.

PFA – Pulverized fuel ash or pulverized fly ash.

Particulate Grout – Any grouting material characterized by undissolved (insoluble) particles in the mix.

Percent Fines – Amount, expressed as a percentage by weight, or a material in aggregate finer than a given sieve, usually the 74 μ sieve.

Permeability – A property of a porous solid that is an index of the rate at which a liquid can flow through the pores.

Phreatic zone – The subsurface zone beneath the water table.

Portland cement – A cement with relatively high strength and slow and even setting.

Pozzolan – Materials, natural or artificial, used as cement replacement, such as PFA or condensed silica fume.

Pumpability – A measure of the properties of a particular grout mix to be pumped as controlled by the equipment being used, the formation being injected, and the engineering objective limitations.

Relative density – The ratio of a substance's weight divided by the weight of the same volume of water at 4° C, also called specific gravity.

Resin – Any polymer, either natural or synthetic, that is a basic material for coatings and plastics. Used in grouting applications as the bonding material between rock bolts and rock.

Rheology - The study of deformation of viscous systems.

Set time (initial/final) - Initial - when cement paste starts to harden and loses its plasticity. Final - when cement paste has hardened and lost all plasticity.

Slurry Grout -A fluid mixture comprising solids such as cement, sand, or clays suspended in water.

Soilcrete -An engineered mixture of cementitious materials with existing soils, for example as created by the jet grouting process.

Solution - A homogeneous molecular mixture of two or more pure substances. A true solution consists of particles less than 10^{-9} m suspended in a fluid.

Standpipe - Grout pipe projecting outside the rock surface and firmly bonded to the hole.

Thixotropy - The characteristic of increasing viscosity of the grout without agitation.

Tremie Pipe -A pipe used to place grout underwater. The pipe is placed to the bottom of the hole. The tremie pipe is always kept in the grout and never allowed to rise above the grout/water interface.

Turbo mixer - A mixer that circulates the grout mix at high speed without shearing.

Unconfined compressive strength (U.C.S.) - The crushing force per unit area of a specimen tested without lateral confined support.

Viscosity - The resistance of fluid to flow, measured in cP (centiPoise) or Ns/m^2.

Void Ratio -The ratio of the volume of voids divided by the volume of solids in a given volume of soil or rock.

Water/cement ratio (w/c ratio) - The proportion of water to cement historically measured by volume in the United States.

Water Table -The upper surface of the groundwater profile in soil or rock in the absence of overlying impermeable strata.

1.4 HISTORICAL OVERVIEW

Several different origins are claimed for the derivation of the word "grout" including the Middle English word "grút," which described coarsely ground meal. This word was later used for porridge and, by analogy, came to be used for liquid mortar of similar consistency.[2, 3] The concept of using "hot Lime liquid" to cement together stones used as the hearting material for church walls was reported upon in 1675. Other possible origins include French, Dutch, Portuguese, and Spanish words, mainly referring to milk or soup. However, it may be correctly asserted that the verb "to grout" dates from the early nineteenth century.[3]

The principal types of geotechnical grouting are shown on figure 1 and are as follows:

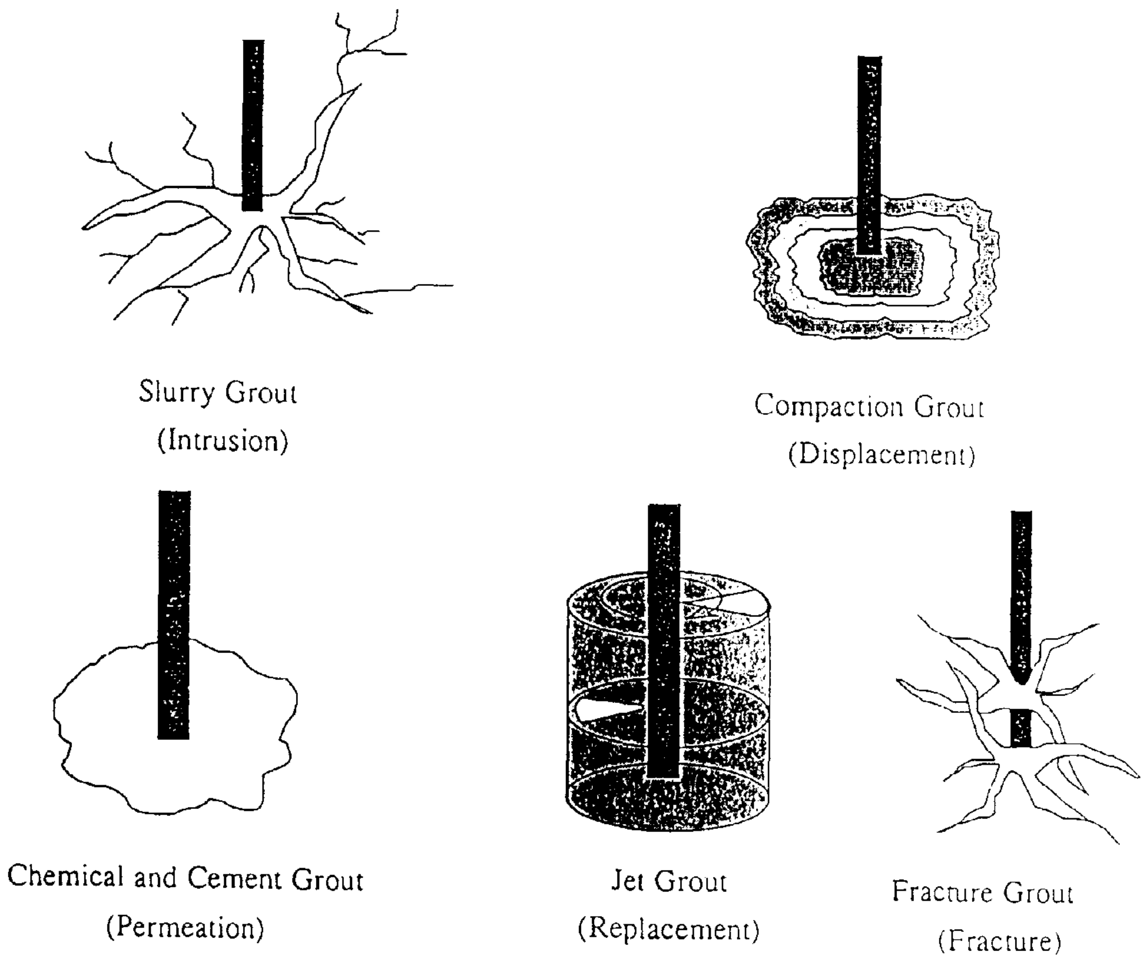


Figure 1. Types of grouting.

1. Rock Grouting
   - Fissures (using slurry grouts)
   - Voids (natural and artificial)

2. Soil Grouting
   - Permeation (using slurry and chemical grouts)
   - Compaction (or displacement)
   - Jet (or replacement)
   - Fracture (including compensation grouting)
   - Lime injection

In the United States, fissure grouting was used first (1890s), followed by chemical grouting and compaction grouting (1950s), jet grouting (1980s), and fracture grouting (1990s).

a. Rock Grouting

Charles Berigny is credited with the invention of pressure grouting in 1802.[3] This system was named the "Injection Process" and utilized excess pressure to pump a suspension of clay and lime to repair deteriorated masonry walls in the city of Dieppe, France. The earliest use of portland cement as a grout is credited to Marc Brunel, who used it on the first Thames Tunnel in England in 1838, and to W.R. Kinnipple who introduced the pressure injection process to England in 1856. In 1876, Thomas Hawksley used cement grouts to inject fissures in rock in England.[4]

Although W.E. Worthen is claimed to have done some masonry pier injection at Westford, CT in 1854, and R.L. Harris constructed grouted concrete foundations in 1891 at Croton Lake, NY, it was not until 1893 that the pressure grouting process appears to have been used systematically to fill cavities (in limestone) under an american structure (New Croton Dam, NY).[5]

There then followed considerable activity with slurry grouts in repairing fissures in masonry bridge piers, and other brick and masonry structures, as well as in underwater applications (preplaced aggregate concrete, and tremied foundations), many of them related to railroad construction.

In 1910, grouting of Estacada Dam, OR, was commenced, believed by the consultants of the project to be the first systematic rock fissure grouting project to have been undertaken in the

United States, with the intention of creating a hydraulic cut-off.[3] This proved to be the forerunner of the intense period of dam construction, and grouting, in the United States which lasted from the 1920s until the 1970s. During this time thousands of projects were executed, largely under rigid "Prescriptive" type specifications to ensure standardization of approach within and between, usually Federal, owner organizations. This goal was achieved, but at the expense of native innovation and the absence of foreign input. As a result by the early 1980s, American practice was certainly different from, and arguably somewhat behind, European practice. However, since then the activities of certain specialty contractors, consultants, and materials and equipment suppliers, and the ever-challenging demands placed on owners principally in the field of dam rehabilitation, have resulted in significant changes. The resulting technical enhancements in techniques and abilities have been fostered by a growing use of "Performance" type, Design-Build, specifications, such as are more common in other countries, and a better understanding of the basic engineering design rationales.[6]

Rock fissure grouting is mainly used to provide hydraulic cut-offs of relatively low permeability, but it can also be used to bind together rock masses mechanically to enhance load bearing properties.

Similar drilling and grouting techniques are also widely used to locate and seal major voids in rock masses. These voids may be naturally created (e.g., karstic limestone features, or salt solution cavities), or can be due to human activities (e.g., mineral workings such as coal mines). Such voids can generate surface settlements, or can permit the relatively easy flow of large volumes of water under hydraulic gradients. The grouting methods and materials used largely depend on the application, and are reviewed in chapter 2. However, it may be observed that for economic reasons alone, various fillers such as fly ash, sand, and gravel are usually incorporated into void filling grouts.

As a subset of void filling operations, the term "slabjacking" (or "mudjacking") is common as illustrated in figure 2. It refers to the pressure injection of slurry grouts of varying consistencies for the purpose of raising and releveling settled concrete pavement or concrete slabs. Slabjacking is also used for under-slab void filling and joint "pumping". There is no one "typical" practice in this field; local "belief" in what is best for the job at hand seems to be the norm. For instance, slabjacking may utilize a variety of fillers ranging from fly ash to lime to hot asphalt, and grout consistencies ranging from very fluid to zero slump. In addition, certain proprietary processes using expanding polyurethane foams to create uplift pressures and generate movements are used.

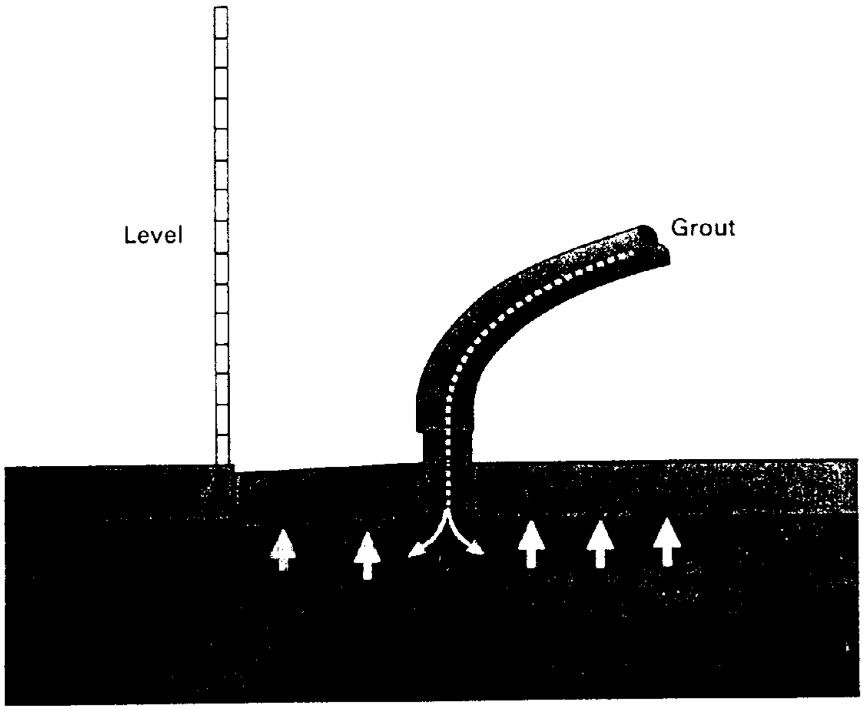


Figure 2. Mudjacking schematic.

Recommended reading in rock grouting include:

- "Construction and Design of Cement Grouting - A Guide to Grouting in Rock Foundations".[3]
- "Dam Foundation Grouting". [5]
- "Grouting of Rock and Soil".[7]
- "Practical Guide to Grouting of Underground Structures". [8]
- ASCE Specialty Conferences in 1982, 1992, 1997 and 1998.[9, 18, 19, 20]

b. Soil Grouting

Permeation Grouting

A variety of materials, particulate, colloidal and solution, can be used to permeate soils, the exact choice largely depending on the grain size distribution (and hence, permeability) of the soil mass as shown in figure 3. Due to their relatively large particle size, conventional portland cements can only permeate into gravels and coarse sands in properly formulated grouts. When attempting to grout finer soils, a filter cake develops at the borehole, preventing further grout permeation. In 1983, ultrafine cement was first introduced into the United States. This led to a new family of fine-grained, fine-ground cements that could be used to permeate finer sands. This process was then taken further with the better understanding of the vital roles of pressure filtration and cohesion, in controlling grout penetrability in the 1990s.[11]

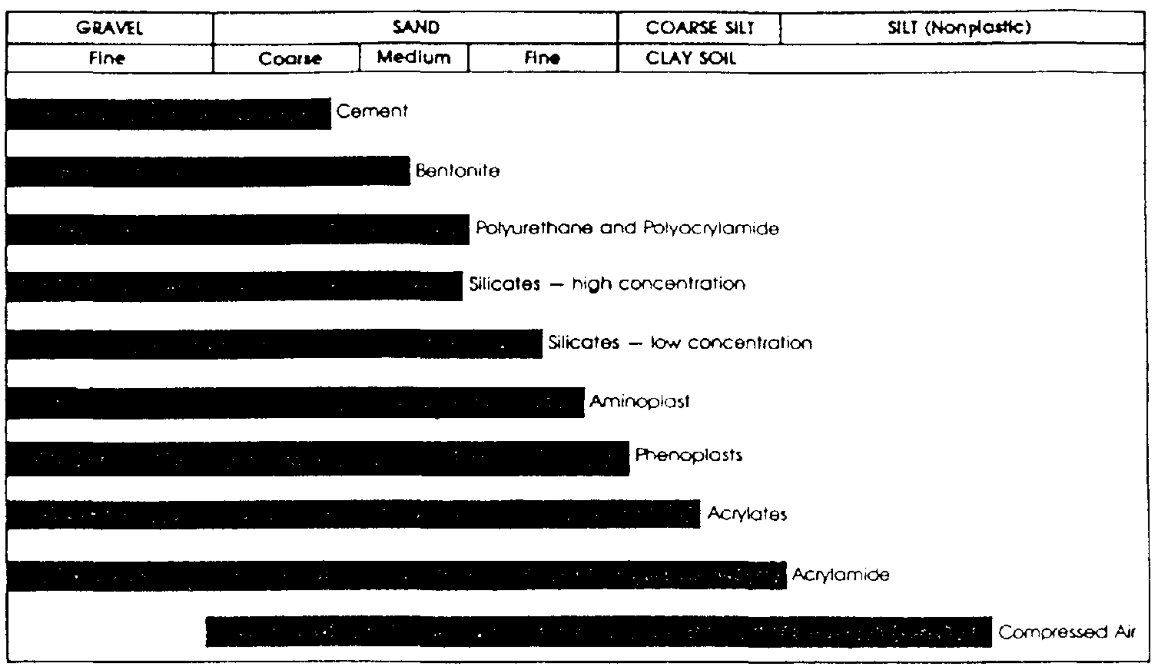


Figure 3. Penetrability of various grouts.[4]

Development of chemical grouting was a natural progression evolving from the limitations of early particulate grouting, such as particle size, setting times, and resistance to flowing water while setting.

The first recorded patent concerning chemical grouting was obtained by Jeziorsky in 1886 and was based on injecting concentrated sodium silicate into one hole and a coagulant into the adjacent hole. H.J. Joosten, a Dutch engineer, demonstrated the reliability of this chemical grouting process in 1925. His system of injecting concentrated sodium silicate during the grout pipe placement and a strong calcium chloride solution during the grout pipe withdrawal is known worldwide as the "Joosten Process". From then until the early 1950s, sodium silicate formed the basis for all chemical grouts.(4)

In the 1950s, advances in polymer chemistry, aimed at reducing the two-step Joosten process to a reliable, single-shot system, resulted in the development of a number of new, proprietary grouts. Two products, one an acrylamide grout and the other a single-shot, silicate based grout coupled with timed gel control, dominated the American market. However, in Japan in 1974, incidents of water poisoning linked to the use of acrylamide grouts led to an immediate ban on acrylamides in that country and subsequently to a ban on all chemical grouting materials except silicate based grouts not containing toxic additives.

At the same time in the United States, environmental pollution prevention was beginning to gain national attention. Prompted perhaps by the Japanese incident, studies were therefore conducted on acrylamide grout. Responding to the concerns being voiced, the major domestic manufacturer of acrylamide grouts voluntarily withdrew the product from the market in 1978, through acrylamides had not been banned, and in fact are still in limited use.

Because a very specialized sewer-sealing industry had grown dependent on the use of acrylamide grouts, those involved in the industry began searching for an alternative. Acrylate grouts, with properties similar to those of acrylamide grouts but environmentally more acceptable, began to emerge as a general replacement for water control.

Sodium silicate based grout is still the most widely used grout for soil stabilization, and indeed it is claimed that "virtually all construction grouting in the United States is done with silicates".(4) The silicate is reacted with either an organic or inorganic reagent, depending on the required gel properties, as described in chapter 2.

Permeation grouting is intended to fill all (or most of) the natural pore spaces in a soil mass, without changing the virgin structure or volume. Grouts can thus be used to increase the cohesion between soil particles, thereby leading to increased strength parameters, and/or reduced permeability. As a general rule, the finer the pores, the higher the cost of the grout, and therefore is normal to attempt to fill larger pores first with particulate grouts, and to permeate into finer or residual pores with chemical grouts.

Recommended reading in permeation grouting include:

- Grouting of Rock and Soil.[7]
- "Grouting in Soils - Volume I - A State of the Art Report; Volume II - Design and Operations Manual", 1976.[10]
- "Chemical Grouts for Soils - Volume I - Available Materials; Volume II - Engineering Evaluation of Available Materials", 1977.[12]
- "Design and Control of Chemical Grouting - Volume I - Construction Control; Volume II - Design Concepts; Volume III - Engineering Practice; Volume IV - Executive Summary".[13]
- "Grouting in the Ground".[14]
- "Verification of Geotechnical Grouting".[15]
- "Grouting and Deep Mixing".[16]
- "Chemical Grouting".[17]
- ASCE Specialty Conferences.[(9,18,19]

These reports and other research were instrumental in the design, specification and utilization of chemical grouting on the Baltimore, Washington, Pittsburgh, Los Angeles, Seattle, and Boston subways.

Compaction Grouting

Like chemical grouting, compaction grouting has also overcome certain problems inherent with the use of fluid, particulate grouts. Compaction grouting was pioneered on the West Coast in the 1950s and is the only grouting technique to have its origins in the United States. It was first used to rectify structural settlements through the controlled injection of a very stiff, low mobility mix.[20] In the late 1970s, compaction grouting was introduced as a preventative rather than a remediation measure when the technique was used in lieu of conventional underpinning to protect surface structures from settlement during the installation of the soft ground Bolton Hill Tunnel, part of the Northwest Line of the Baltimore Region Rapid Transit System.[21]

The recognition that potentially liquefiable soils can be densified by compaction grouting led to test programs to verify that such loose soils beneath structures could be adequately improved by this grouting technique. The West Pinopolis Dam Test Program in 1985 showed that a compaction grouting program can be designed to obtain the level of densification required at a specific site to improve the seismic stability in situ, provide recommendations to monitor the results, and verify the potential economics of this system.[22, 23]

Since the 1980s, compaction grouting has also been used to rectify karst-related subsidence under both new and existing structures in limestone terrains.[24, 25]

Compaction grouting features the use of low slump (usually 25 mm or less), low mobility grouts of high internal friction. In weak or loose soils, the grout typically forms a coherent "bulb" at the tip of the injection pipe thus compacting and/or densifying the soil around.

When injected into loosened areas above tunnels or sinkholes compaction grouting will redensify the soil and thereby prevent surficial settlement. If settlement has already occurred, careful compaction grouting may be used to lift and level any surface structures which have been impacted.

Compaction grouts can be designed as an economic and controllable medium for filling large voids, even in the presence of flowing water.[50]

Recommended reading in compaction grouting include:

- "Compaction Grouting," Chapter 7, Ground Improvement.[26]
- "Compaction Grouting – The First Thirty Years".[20]
- "Compaction Grouting to Control Ground Movement During Tunneling".[21]
- "Grouting: Compaction, Remediation and Testing".[27]

- "Compaction Grouting – State of Practice 1997".[28]

Jet Grouting

Jet grouting was developed in Japan in the early 1970s[29] based on a British concept dating from the 1960s. Since its reintroduction into Europe in the latter part of the 1970s it has been used extensively for underpinning and/or excavation support of sensitive structures, groundwater cut-off control, and tunneling applications.[30, 31, 32] In the early 1980s in the United States, jet grouting utilizing conventional drilling and grouting equipment was tried on a few demonstration projects. This equipment proved to be ineffective and jet grouting underwent a hiatus until 1987, when it was reintroduced using equipment specifically designed for the technique and incorporating contemporary European equipment and knowledge. The combination of sophisticated equipment, more extensive technical knowledge, and proper applications make this a successful ground treatment technique, usable with almost any soil type. This success is demonstrated by over 200 projects having been completed between 1988 and 1997 in the United States.

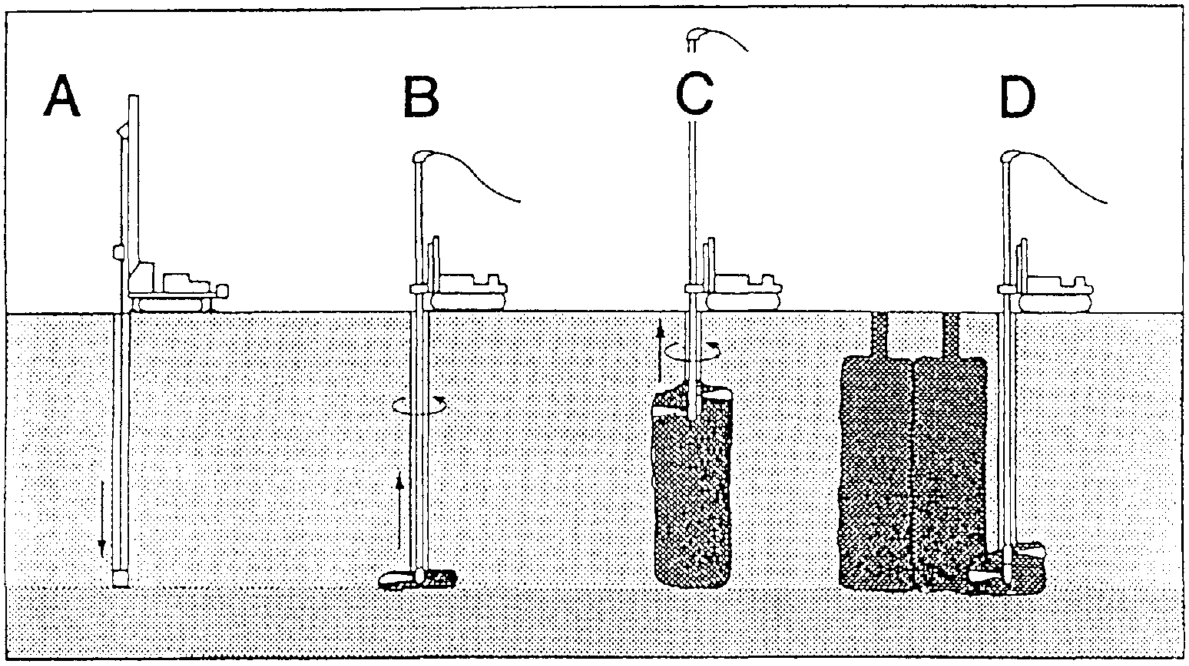
The different types of jet grouting are intended to transform soils into a mixture of soil and cement, typically referred to as "soilcrete". Jet grouting permits the shape, size and properties of these treated masses, usually circular columns, to be engineered in advance, with an increasingly high degree of precision as illustrated in figure 4. There are basically three distinct types of jet grouting as schematically shown on figure 5:

- One Fluid System: The fluid is grout, and in this system the high pressure (up to 50 MPa) jet simultaneously erodes and injects. It involves only partial replacement of the soil.

- Two Fluid System: This method uses the high pressure cement jet inside a compressed air cone. This system gives a larger column diameter than the one-fluid system and gives a higher degree of soil replacement.

- Three-Fluid System: Here, an upper ejection of high pressure water (30 to 50 MPa) inside compressed air envelope is used for excavation, with a lower jet (usually at lower pressure) emitting grout to replace the jetted soil.

Jet grouting has the potential to treat the whole spectrum of soils, from sands and gravels to highly sensitive clays. The details of the results obtained will reflect the operational parameters used, and the nature of the virgin soil, as identified in chapter 3.



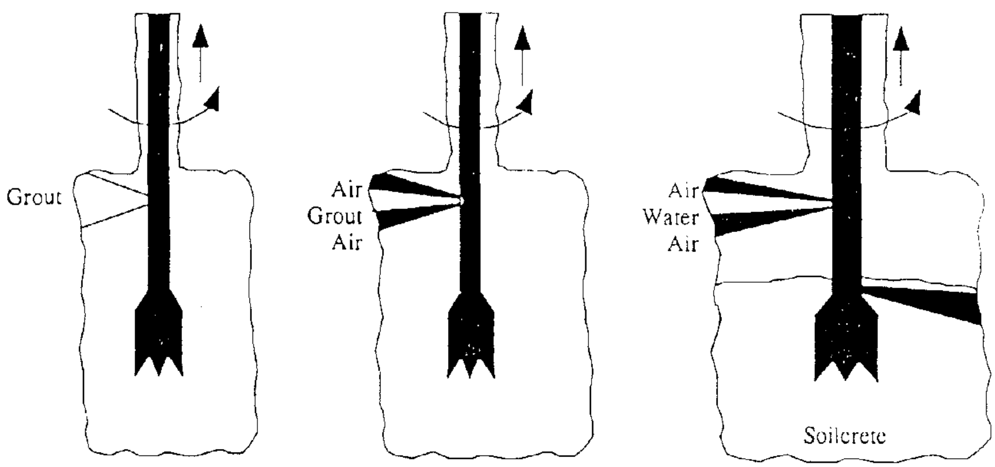
A Drilling

B Erosion and mix-in-place operation

C Development of column-like element

D Completed elements forming a wall-like structure of interlocking elements

Figure 4. Principle of jet grouting operation.[7]



(a) Single fluid(s) (b) Double fluid (D) (c) Triple fluid (T)

Figure 5. Different systems of jet grouting.

Recommended reading in jet grouting include:

- "Jet Grouting - Uses for Soil Improvement".[30]
- "Jet Grouting Ground Improvement".[31]
- "Jet Grouting, Ground Control and Improvement".[32]
- "Grouting of Rock and Soil".[33]

Soil Fracture Grouting

In the course of otherwise routine permeation grouting activities, it was often observed that sheets or lenses of grout could be induced to travel away from the point of injection using certain combinations of material and injection parameters. Such soil fractures could therefore be used to improve the overall performance of soil masses by providing a stiff "internal" grout skeleton. Developments in France in the 1970s led to the concept of using carefully controlled fracturing of the soil to compensate for surface settlements caused by underground tunneling. By the 1990s, "compensation grouting" or soil fracture grouting, using sophisticated construction and monitoring equipment was being used in urban areas subject to soft ground tunneling (e.g., London's new Jubilee Line Extension) and tunnels in Sarnia, Ontario.[34] Most recently, the technology has been applied for a similar application for the new Metro in San Juan, Puerto Rico.

On the West Coast, less sophisticated "lense grouting" as shown on figure 6 had been undertaken for slope stabilization since the late 1980s.[33] Specially formulated high rheology particulate grouts are injected repeatedly through arrays of grout pipes, the exact parameters being controlled in response to the desired surface response characteristics. Extremely careful control is exercised over the process so that the greatest benefits can be realized in terms of surface movements.

Recommended reading in soil fracture grouting includes:

- "Soil Fracture Techniques for Terminating Settlements and Restoring Levels of Buildings and Structures," Ground Improvement[35]
- "Lense Grouting with Fiber Admixtures to Reinforce Soils".[33]

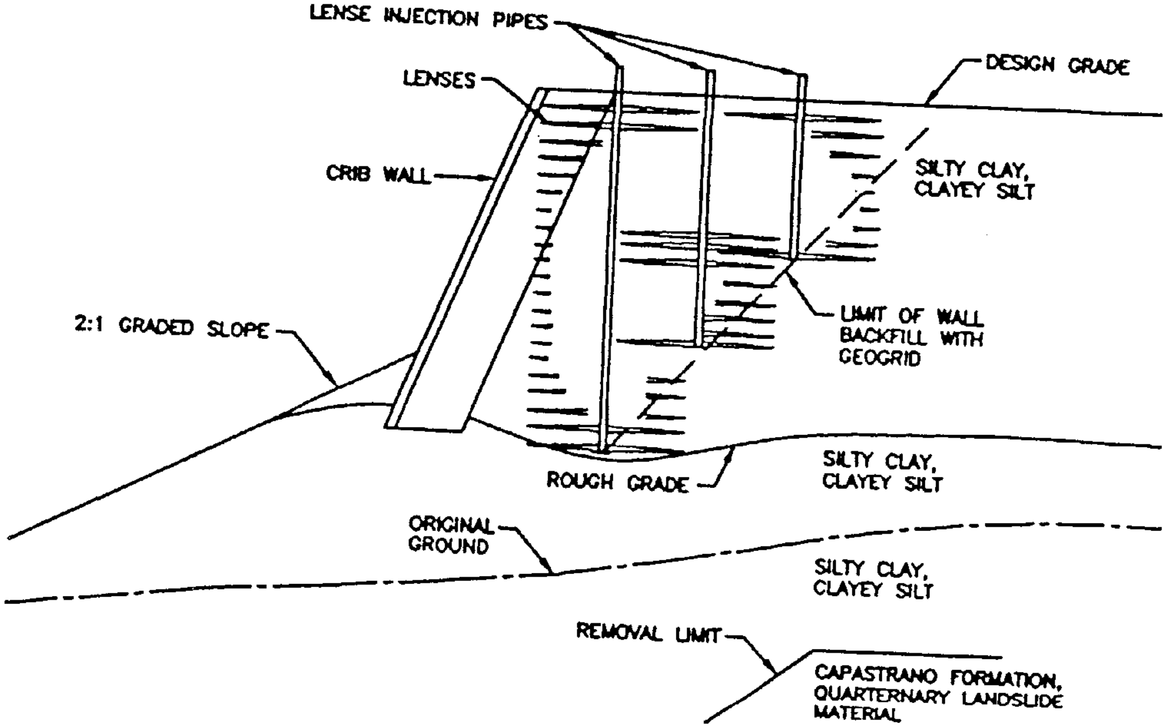


Figure 6. Lense grouting application.[33]

Lime Injection

In cohesive soil, where fluctuations in the moisture content of the ground create cyclical expansion and shrinkage, severe damage can result, such as slope failure, large and small spider web-like cracks in pavement/concrete, cracked masonry, and structural movement. Particularly in Texas, lime injection grouting has been used for many years to rectify these situations, either by strengthening the underlying soil to correct an existing problem or as a pretreatment to limit or eliminate expansion and shrinkage of the soils.

Hydrated lime slurries, usually containing a filler such as fly ash, is injected into the expansive clay soils to typical depths of 0.9 to 3.0 m. The grout then follows the path of cracks and joints, creating a rigid skeleton of lime seams and strengthened soil layers. This strengthening effect is due to the pozzolanic reaction of the lime with the clay. Using fly ash or some other inexpensive filler helps to fill the gaps where moisture once resided, adding additional bulk and strength. Once the grout has set, movement of both moisture and soil are inhibited.

CHAPTER 2

DRILLING AND GROUTING PRINCIPLES, TECHNIQUES AND MATERIALS

A wide range of grouting techniques are used, reflecting the wide spectrum of ground conditions and applications. Although there are certain elements of practice that are technique specific, there are other aspects which are universal. For example, each technique requires the drilling of holes in the ground, and the controlled injection of a grout formulated from a well-defined range of products. The technique specific elements include the equipment, hole spacing, construction sequencing, and refusal criteria. Whereas these aspects are detailed for each technique in subsequent chapters, this chapter provides background to the commonalities.

2.1 FUNDAMENTALS OF PROGRAM DESIGN

A grouting program may be developed for implementation under a performance specification or under a method (prescriptive) specification.

Design under a performance specification generally leaves to the specialty contractor the responsibility for developing specific methods and quantities, depending on the intent of the work, and imposes well-defined performance requirements such as:

- Increase of density in a treated area to a predetermined minimum as determined by in situ sampling and testing methods (e.g., SPT, CPT, pressuremeter, etc.).
- Increase in strength as measured by unconfined compression testing, or plate loading tests.
- Decrease in permeability as determined by in situ permeability testing or seepage absorption, or seepage measurements into excavations, piezometers, etc.
- Settlement restrictions/tolerances.

Designs under a method specification require extensive and detailed knowledge of grouting by the specifiers, with respect to materials, methods, equipment, and performance monitoring and verification.

Under either method, quantity estimates are necessary to establish budget costs and/or construction schedules.

For either approach and for most grouting projects, a primary grout hole spacing is selected based on the project requirements and a study of available subsurface information. Primary spacing should be sufficiently wide such that connections between individual grout holes do not normally occur. In practice, primary grout hole spacings vary from 3 to 15 m but there is no "universal", constant distance. Primary holes in effect act as another phase of exploration. Secondary grout hole spacings split the spacing of the primary holes. Large grout takes at secondary locations are a good indicator of the need for additional holes, either generally, or in specific zones. The process continues through intermediate tertiary and higher order holes until analysis of the data confirms that the project goals appear to have been met. The last sequence of holes installed effectively act as verification holes, to supplement any other post grouting investigation studies.

A minimum drill hole diameter is normally specified for grout holes. Many rock grouting projects have been successfully accomplished with grout holes as small as 38 mm in diameter. However, for deep, inclined grout holes, larger diameter grout holes are frequently required which require large diameter drill rods which are more rigid, resulting a straighter bore. This is important at depths greater than 60 m, where drift can otherwise be substantial. Grout holes over 150 mm in diameter are rare. The overall program design concept is therefore implemented in three phases:

1. Exploration, and site characterization.
2. Production, with real time monitoring and data analysis.
3. Verification of performance via late order holes, and other appropriate investigations.

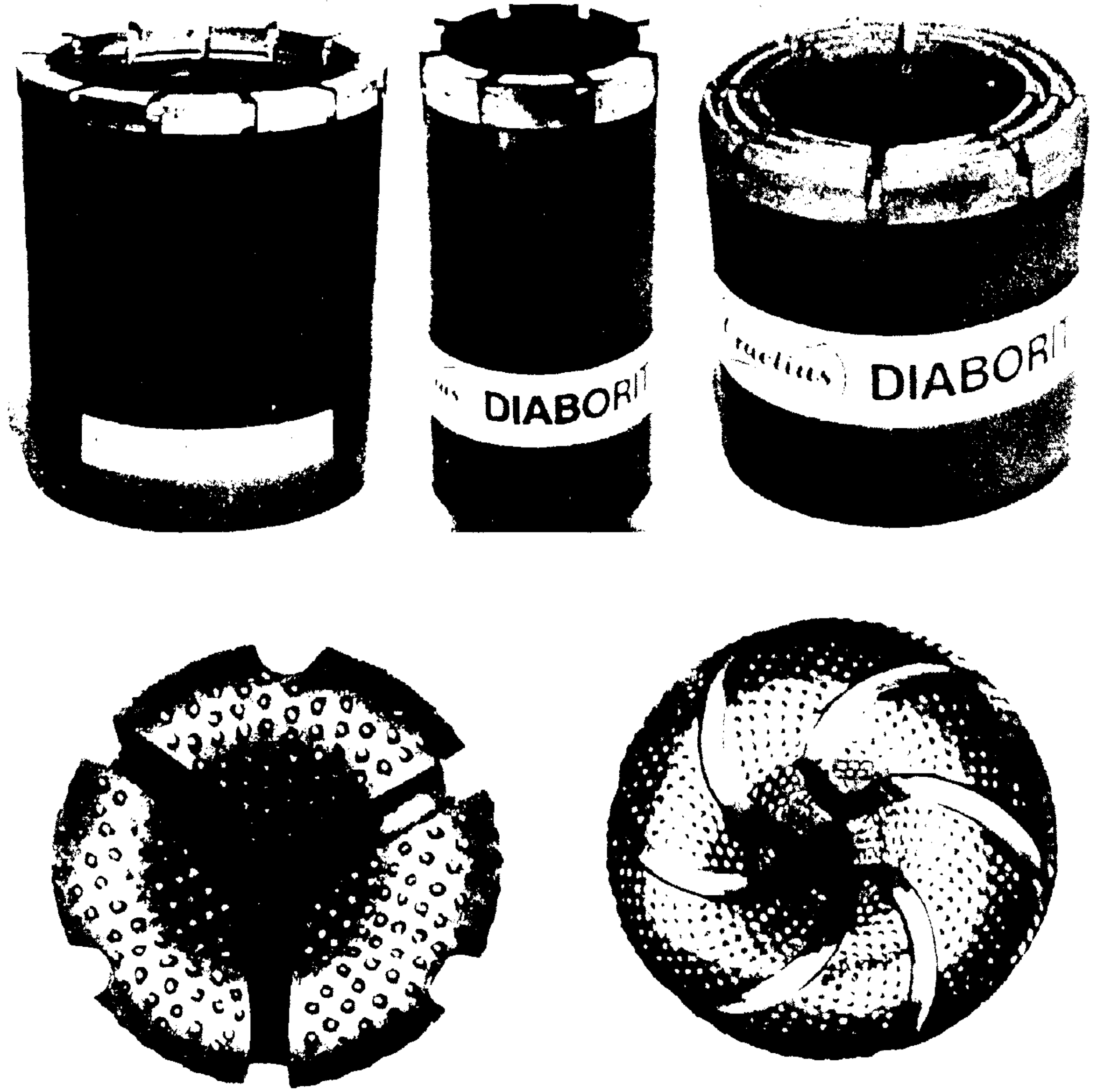
2.2 DRILLING METHODS, EQUIPMENT

These drilling methods are applicable for all three phases of the grouting program.

a. Rock Drilling Methods

There are basically three methods of rock drilling:

1. *High Rotation Speed (i.e., ≥ 600 rpm) / Low Torque Rotary*: relatively light drill rigs can be used to extract core samples, when using a core barrel system, or can also be used simply to drill grout holes, using "blind" or "plug" diamond impregnated bits as illustrated in figure 7. Typically used for holes up to 76 mm diameter to depths of 50 to 150 m.



Top from left: Surface set core bit, impregnated core bit, reaming shell (courtesy of AC)
Bottom: Solid bits[7]

Figure 7. Diamond drilling tools.

Advantages of high speed rotary drilling include:

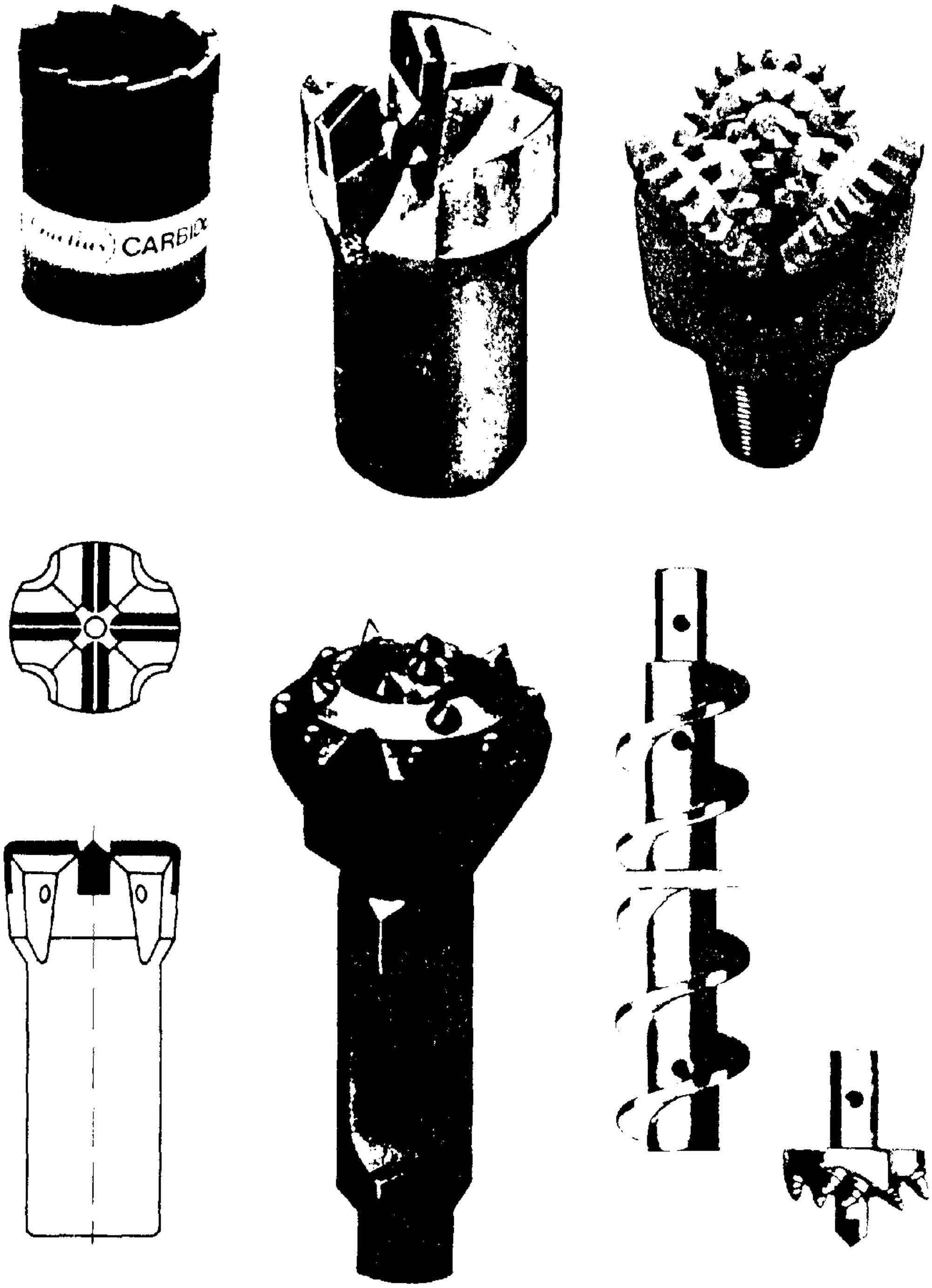
- The same equipment can be used for both investigatory and grout hole drilling.
- Continuous or intermittent exploration of the rock is possible over the entire length of the hole.
- Drilling can be done to relatively great depths.
- Straight holes are drilled with only little deviation.
- No or limited clogging of the rock fissures typically occurs. Cuttings are removed from the hole with the flush water.
- It is possible to drill in all kinds of rock.
- It is possible to use most power alternatives to drive the equipment (i.e., air, electricity, diesel).
- Rotary drill bits produce smooth hole walls which make subsequent packer installation easier.
- Good penetration speeds can be achieved in soft formations.

2. *Low rotational speed / high torque rotary*: used with heavier and more powerful rigs to drill holes of greater diameter to considerable depths. The penetration rate also depends on the amount of thrust applied to the bit. A variety of drag, roller, or finger bits are shown on figure 8.

3. *Rotary percussive*: the drill bit (cross or button) is both percussed and rotated. In general the percussive energy determines the penetration rate either with a top hammer where the drill rods are rotated and percussed by the drill head on the rig or with a down-the-hole hammer: where the (larger diameter) drill rods are only rotated by the drill head, and compressed air is fed down the rods to activate the percussive hammer mounted directly above the bit.

Top hammer drilling is performed at rotation speeds of approximately 80 to 160 rpm in hole diameters seldom above 102 mm. Hole depth is limited to approximately 60 m by power, and by hole deviation concerns.

Down-the-hole drilling is performed at approximately 10 to 60 rpm in hole diameters of 85 mm and above to depths of over 100 m.



Top from left (rotary): Core bit (AC), single-stage bit (Krupp Widia), roller bit (Christensen)
Bottom from left (percussion): Cross bit (Krupp Widia), button bit (Krupp Widia), rotary endless auger (HÜtte)

Figure 8. Carbide drilling tools.[7]

Percussion drilled grouting holes should be flushed by water to avoid the cuttings clogging the fissures. Especially below the water table, air flushing is risky, as a sludge may be formed that closes off the fissures that will have to be grouted at a later stage.

Advantages of percussion drilled grout holes include:

- Higher and consistent penetration rates in rock than for other methods.
- Small and light drill rigs can be used; these are easily moved from hole to hole on the surface.
- Low drilling costs compared with rotary drilling.
- It is possible to optimize the equipment for drilling through layers of different hardness and thickness.

Top hammer drilling is the most common and generally also the least expensive method, but it also limits the hole depth and is subject to the greatest hole deviations. This means increased numbers of holes and costs as well as lower quality. Down-the-hole hammer drilling gives straighter and deeper holes.

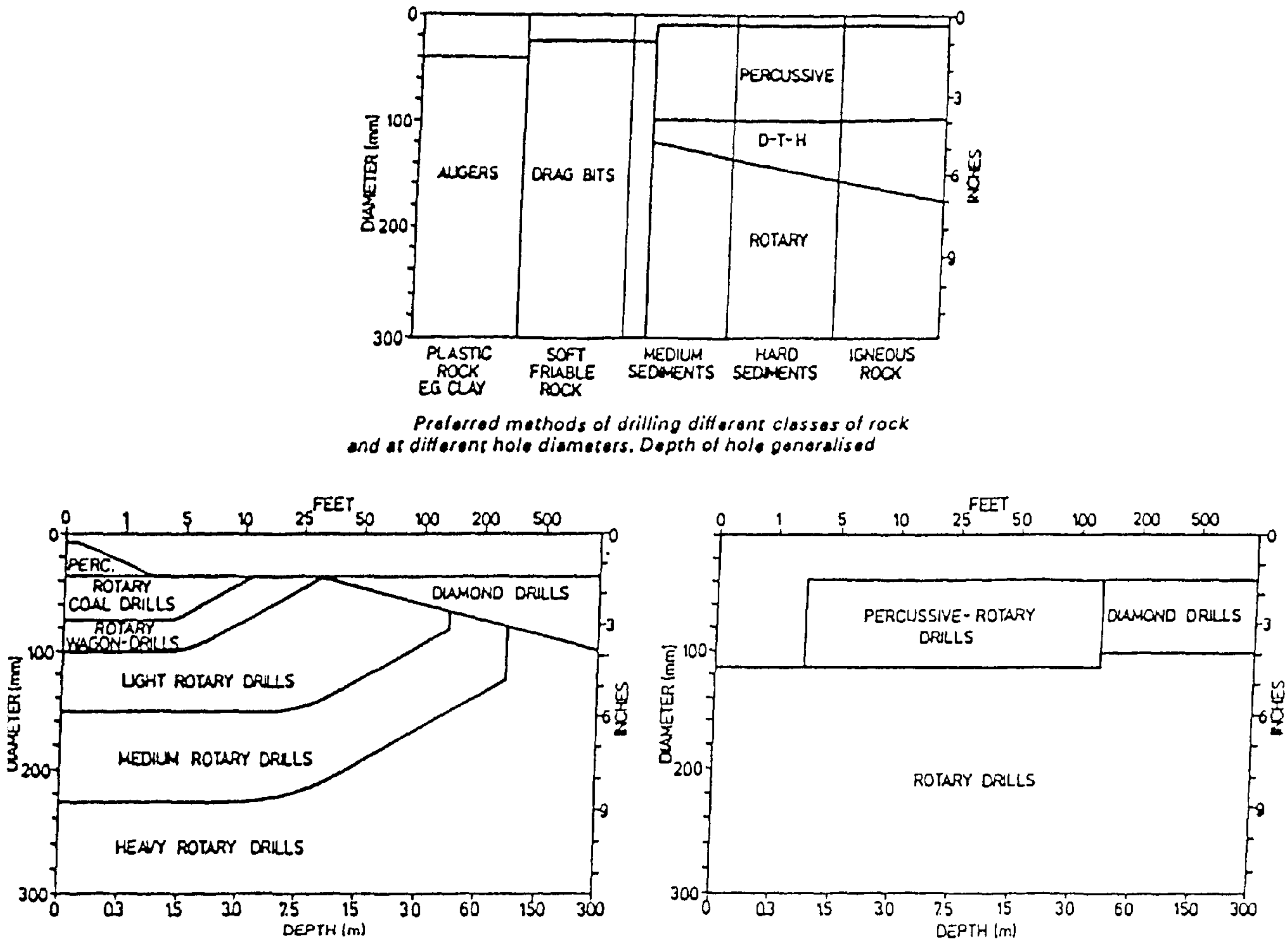
In principle, the prime controls over the choice of drilling method should ideally be related to the geology, the hole depth, and diameter as indicated in figure 9. Hole linearity and drill access restraints may also have significant impact on choice.

In the United States, rock drilling is largely and traditionally conducted by rotary methods although the insistence on diamond drilling is no longer so prevalent. However, top drive rotary percussion is growing in acceptance due to the increasing availability of higher powered diesel and hydraulic drill rigs using water or foam flush. Air flush methods are applicable for drilling grout holes to locate and fill large voids such as karstic features.

Summary, rock drilling

The drilling method selected must:

- Drill a straight hole;
- Protect the hole walls from caving in;
- Produce drill cuttings of such a size, that they can be flushed out without closing the fissures in the ground or blocking the subsequent grouting; and
- Be as economical as possible.



Preferred methods of drilling different classes of rock and at different hole diameters. Depth of hole generalised

Preferred methods in soft friable rocks

Preferred methods in variable strata

Figure 9. General guides for selecting drilling method and equipment for rock drilling.[53]

Where possible, grout holes should be drilled at right angles to the main rock mass fissures, in order to intercept as many as possible. Where this requirement is difficult to meet, the spacing must be reduced instead to ensure that fissure planes with an unfavorable orientation to the grout holes will be grouted as efficiently as possible.

Larger diameters cores provide more information about the ground. Because of the stiffness of the drill string, larger hole diameters in general result in straighter, but more expensive holes.

The setting of packers is more expensive, and also more difficult in larger diameter holes, and final backfilling costs higher.

Hole straightness is important to address, since excessive deviation may leave unpenetrated "windows" in the curtain leading to incomplete treatment.

For greater hole depths, guide rods (centralizers) and drill string supports may be used, together with thicker walled drill rods.

Some commonly attainable hole deviation limits are:

- High speed rotary drilling: normally 2 to 5% to depths of 80 m.
- Top hammer drilling: long holes - 15 to 20% (with guide rods, under 5% can be reached); shallow holes, down to 12 to 15 m - under 5% is possible also without guide rods. Long top hammer holes drilled with guide rods incur a high risk of getting stuck.
- Down-the-hole drilling: typically less than 2%, and less than 1% with high standards of workmanship.

The size of the drill cuttings can vary from a muddy clay to flaky gravel. Different drilling methods produce cuttings that vary in form, size, and shape. All holes drilled for grouting must be cleaned carefully of drill cuttings and loose ground material lodged in the cracks. In general, this is done by high pressure water flushing from the bottom up towards the collar of the hole.

Measurement While Drilling (MWD)

MWD is a method for continuous registration and recording of various drilling parameters. It measures the drill rig's behavior during the drilling operation. The measured variables can be depth, rate of penetration (ROP), weight on bit (WOB), feed force, rpm, torque, percussion hammer pressure, flush water flow, flush water pressure, acceleration, and time. MWD is usable both on percussive and rotary drill rigs.

Depending on the ground there will be a variation in the drilling parameters that are registered, mainly the variation of the hydraulic flow and pressure parameters, due to geological variations. Various geological conditions can therefore produce similar hydraulic characteristics. The measured parameters should be correlated with the drilled core sample from a drill hole nearby.

The variables, feed force and rpm are set by the driller. The variables ROP, torque, and acceleration are dependent on the formation being drilled. The variables flush water flow and flush water pressure are dependent on the driller, the drill equipment and the formation being drilled.

For example, the dividend of the flush water flow and the flush water pressure can be used for location of fissures and cracks. The reason is that when the drill bit hits a fissure the pressure will drop and at the same time the flow will increase. This is due to the inflow of the flush water into the fissure. The data may be electronically generated indicating a lag or similar data may be recorded manually and is always of great value in helping to understand the ground, and the changes being effected on it by each successive phase of drilling and grouting.

b. Soil Drilling Methods

There are six commercial generic techniques for grout hole drilling, and the use of bentonite slurry supported open holes. As summarized in table 1 and figure 10 these are:

Single tube advancement: This is the most simple principle. In the drive drilling variant, the casing is percussed and pushed into the soil, without flush, and with a "knock off" disposable bit. With external flush, the casing terminates in an open shoe or "crown" and is rotated into the soil using a strong flushing medium (usually water). The flush emerges from the crown and normally travels to the surface between the casing and the soil.

Rotary Duplex: The term "duplex" means the simultaneous advancement of an outer casing (with crown) and inner drill rod (with bit). The flush is passed down the drill rod, but then is allowed to emerge to the surface through the annulus between rod and casing. In this particular category, the rods and casings are simultaneously rotated in the same direction.

Table 1. Overburden drilling methods.[36]

DRILLING METHOD	PRINCIPLE	COMMON DIAMETERS AND DEPTHS	NOTES
1. Single tube advancement			
a. Drive drilling	Casing, with "lost point" percussed without flush.	50 - 100 mm to 30 m	Obstructions or very dense soils problematical.
b. External flush	Casing, with shoe, rotated with strong water flush.	100 - 200 mm to 60 m	Very common for anchor installation. Needs high torque and powerful flush pump.
2. Rotary duplex	Simultaneous rotation and advancement of casing plus internal rod, carrying flush.	100 - 200 mm to 70 m	Used only in very sensitive soil/site conditions. Needs positive flush return. Needs high torque.
3. Rotary percussive concentric duplex	As 2, above, except casing and rods percussed as well as rotated.	89 - 175 mm to 40 m	Useful in obstructed/bouldery conditions. Needs powerful top rotary percussive hammer.
4. Rotary percussive eccentric duplex	As 2, except eccentric bit on rod cuts oversized hole to ease casing advance.	89 - 200 mm to 60 m	Somewhat obsolescent and technically difficult system for variable overburden.
5. "Double head" duplex	As 2 or 3, except casing and rods rotate in opposite directions.	100 - 150 mm to 60 m	Powerful, new system for fast, straight drilling in very difficult soils.
6. Hollow stem auger	Auger rotated to depth to permit subsequent introduction of reinforcement through stem.	150 - 400 mm to 30 m	Obstructions problematical; care must be exercised in cohesionless soils to avoid cavitation and/or loosening. Prevents application of higher grout pressures.

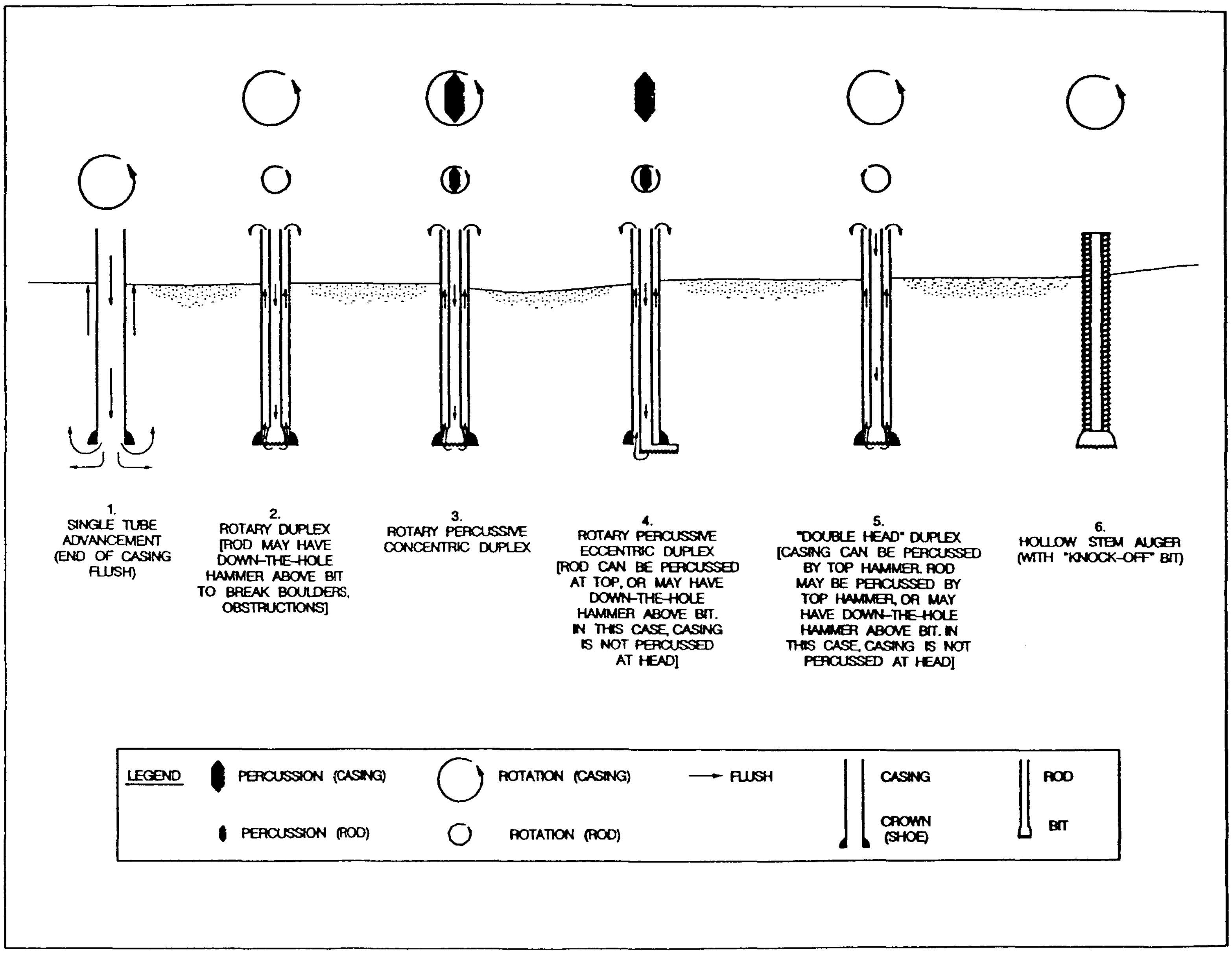


Figure 10. Schematic representation of six generic overburden drilling methods.[36]

Rotary Percussive Duplex (Concentric): Similar to Rotary Duplex except that the rods are also percussed. When a top hammer is used, the casings are simultaneous percussed, whereas if a down-the-hole hammer is used, only a drill bit experiences the percussive action, and the casing is merely rotated.

Rotary Percussive Duplex (Eccentric): Similar to Percussive Duplex except that an eccentric drill bit reamer device on the rod string is used to oversize the hole, to permit the casing to follow without rotation. After the duplex has reached target depth, the reamer is retracted into the casing so permitting the extraction of the rods. Both top drive and down-the-hole versions are available.

"Double Head" Duplex: Similar to Rotary Duplex and Percussive Duplex except that the rods and casings are rotated and advanced simultaneously but in opposite directions. This maximizes the penetration action for any given rig energy, and encourages hole straightness. It is especially useful in very difficult ground conditions. Pure rotary, top drive or down-the-hole rotary-percussive options can be employed.

Hollow Stem Auger: High torque and thrust are used to advance a continuous screw with a hollow core (protected during penetration by a bottom plug). This is a traditional method of drilling cohesive soils and soft argillaceous rocks, and can also be used in coarser materials provided there is significant cohesion and/or cementation.

The logic of choice is perhaps even more obscure than in rock drilling, and history and habit have ensured that not all methods are used by any one contractor, or in any one geographical region. Hollow stem augers are common around the Great Lakes and on the West Coast, while simple flushed casings and rotary duplex are favored in the East. The emergence of foreign-backed drill rental companies offering percussive duplex and double-head duplex capabilities has spread these techniques nationwide. Percussive duplex (eccentric) is in general decline for routine production grout holes, although it is still regarded in certain quarters as the premier soil drilling method, in very difficult conditions.

2.3 GROUTING MATERIALS

There are four categories of materials, listed in order of increasing rheological performance and cost [37]:

1. Particulate (suspension or cementitious) grouts, having a Binghamian performance.
2. Colloidal solutions, which are evolutive Newtonian fluids in which viscosity increases with time.
3. Pure solutions, being non evolutive Newtonian solutions in which viscosity is essentially constant until setting, within an adjustable period.
4. "Miscellaneous" materials.

Category 1 comprises mixtures of water and one or several particulate solids such as cement, flyash, clays, or sand. Such mixes, depending on their composition, may prove to be stable (i.e., having minimal bleeding) or unstable, when left at rest. Stable, thixotropic grouts have both cohesion and plastic viscosity increasing with time at a rate that may be considerably accelerated under pressure.

Category 2 and 3 grouts are now commonly referred to as solution or chemical grouts and are typically subdivided on the basis of their component chemistries, for example, silicate based (Category 2), or resins (Category 3). The outstanding rheological properties of certain Category 3 grouts, together with their low viscosities, permit permeation of soils as fine as silty sands ($k = 10^{-6}$ m/s).

Category 4 comprises a wide range of relatively exotic grout materials, which have been used relatively infrequently, and only in certain industries and markets. Nevertheless, their importance is growing due to the high performance standards which can be achieved when they are correctly used. The current renaissance in the use of hot bitumen grouts for fast flow sealing is a good example.

A summary of some characteristics and material costs is provided in table 2.

1. Particulate Grouts

Due to their basic characteristics, and relative economy, these grouts remain the most commonly used for both routine waterproofing and ground strengthening. The water to solids ratio is the prime determinant of their properties and basic characteristics such as stability, fluidity, rheology, strength, and durability. The following broad subcategories can be identified:

- Neat cement grouts.

Table 2. Characteristics of grout material for water control purpose.

Description	Viscosity (cp water cement ratio)	Toxicity	Strength	Relative Material Cost/Liter	Remarks
PARTICULATE GROUT:					
Type I Cement	High (50cps-2:1)	Low	High	>$0.04	Non-flexible; penetrates only larger cracks
Type III Cement	Med (15 cps-2:1)	Low	High	>$0.05	Non-flexible; penetrates only larger cracks
Microfine Cement	Low (8cps-2:1)	Low	High	$0.30	Non-flexible; penetrates fine cracks
Microfine Cement/Silicates	Low (10cps-3:1)	Low	Med	>$0.17	Non-flexible; penetrates fine cracks
COLLOIDAL SOLUTION:					
Silicates	Low (>6cps)	Low	Med	>$0.13	Penetrates fine cracks
SOLUTION GROUT:					
Lignosulfites	Med (>8cps)	High	Low	>$0.26	Flexible; penetrates fine cracks
Polyrethane	High (>400 cps)	High	High	>$1.32	Flexible; penetrates large cracks
Acrylamides	Low (1.2cps)	High	Low	>$0.53	Flexible; penetrates very fine cracks
Acrylates	Low (1.5cps)	Low	Low	>$0.53	Flexible; penetrates very fine cracks

- Clay/bentonite-cement grouts.
- Grouts with fillers.
- Grouts for special applications.
- Grouts with enhanced penetrability.

Typically in the United States, water/cement (w/c) ratios have been expressed as a volumetric ratio. Given the increased use of semi-automatic batching equipment, it is easier to work in weight ratios. For example, a grout of w/c = 1 by weight comprises 50 kg of water (50 L) and 50 kg of cement. Additives and admixtures are normally expressed also as a weight ratio to cement. As a rule of thumb, to obtain water/cement ratios by volume, multiply the water/cement ratio (by weight) by 1.5.

Portland cements are the most common and best known cements used worldwide as the basic ingredient for particulate grouts. The following provides a general description:

Type I portland cement is accepted as the general purpose cement for use in the majority of grouting applications when the special properties of other types are not required.
Type II portland cement is manufactured to resist moderate sulfate attack and to generate a slower rate of heat of hydration than that exhibited by Type I.
Type III portland cement is used when higher early strengths are desired. It is considered for phases of grouting applications to be put into service quickly or for emergency repairs. Since particle size is smaller than in other types, it is sometimes specified for grouting slightly finer fissures.
Type IV portland cement generates less heat during hydration than Type II and develops strength at a much slower rate that Type I. It is considered for use in large, mass grout placements when high temperatures of heat of hydration are not acceptable.
Type V portland cement is manufactured for use in grout exposed to severe sulfate action. It is used principally when a high sulfate content is present in soils or groundwaters.

Microfine cements are simply finer ground versions of both portland and blast furnace slag cements. Typically the maximum particle size is less than 8 μ, with the bulk being less than 4 μ. Examples of the gradation curves from some of the many types now available in the U.S. are shown on figure 11.

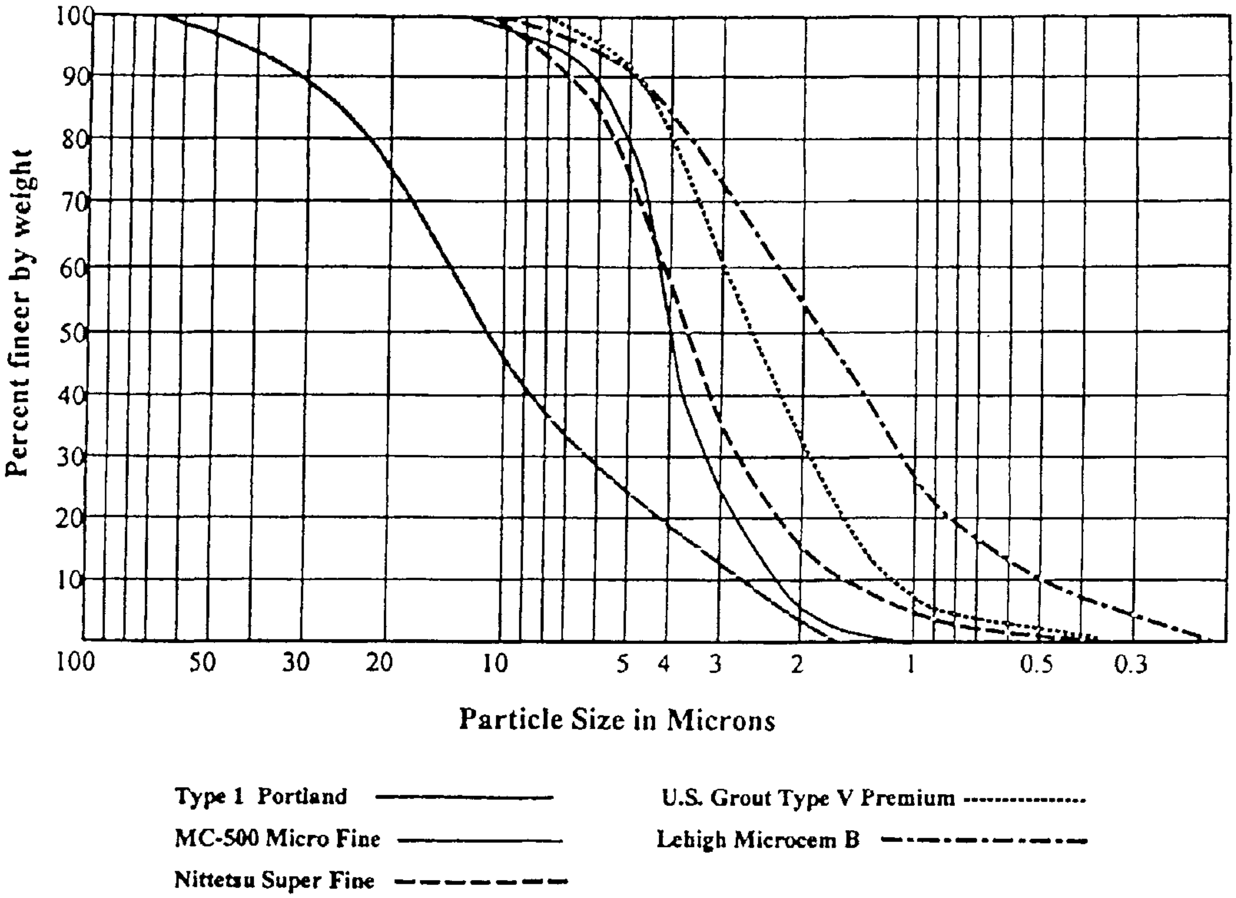


Figure 11. Grain size distribution for various cements.[11]

Note that many particulate grouts are unsuited for sealing high flow, high head conditions, they will be diluted or washed away prior to setting in the desired location. Low mobility grouts ("compaction grouts") can be classified in the third subgroup, and can be used for seepage reduction under appropriate conditions

2. Colloidal Solutions

Colloidal solutions comprise mixtures of sodium silicate and reagent solutions, which change in viscosity over time to produce a gel. Sodium silicate is an alkaline, colloidal aqueous solution. It is characterized by the molecular ratio R_p, and its specific density, expressed in degrees Baumé (Bé). Typically R_p is in the range 3 to 4, while specific density varies from 30 to 42 Bé. Reagents may be organic or inorganic (mineral). The former cause a saponification hydraulic reaction that frees acids, and can produce either soft or hard gels depending on silicate and reagent concentrations. Common types include monoesters, diesters, triesters, and aldehydes, while organic acids (e.g., citric) and esters are now much less common. Inorganic reagents contain cations capable of neutralizing silicate alkalinity. In order to obtain a satisfactory hardening time, the silicate must be strongly diluted, and so these gels are typically weak and therefore of use only for waterproofing. Typical inorganic reagents are sodium bicarbonate and sodium aluminate.

The relative proportions of silicate and reagent will determine by their own chemistry and concentration the desired short- and long-term properties such as gel setting time, viscosity, strength, syneresis, and durability, as well as cost and environmental acceptability.

In general, sodium silicate grouts are unsuitable for providing permanent seepage barriers against high flow/high head conditions because of their relatively long setting time (20 to 60 minutes), low strength (less than 1 MPa) and poor durability. Sodium silicate solution without reagent may be used to accelerate the stiffening of cementitious grouts, a traditional use against fast flows.

3. Pure Solutions

Resins are solutions of organic products in water, or a nonaqueous solvent, capable of causing the formation of a gel with specific mechanical properties under normal temperature conditions and in a closed environment. They exist in the following forms characterized by their mode of reaction or hardening:

- Polymerization: activated by the addition of a catalyzing element (e.g., poly-acrylamide resins).
- Polymerization and Polycondensation: arising from the combination of two components (e.g., epoxies, aminoplasts).

In general, setting time is controlled by varying the proportions of reagents or components. Resins are used when particulate grouts or colloidal solutions prove inadequate, for example when the following grout properties are needed:

- particularly low viscosity.
- very fast gain of strength (a few hours).
- variable setting time (few seconds to several hours).
- superior chemical resistance.
- special rheological properties (pseudoplastic).

- resistance to high groundwater flows.

Resins are used for both strengthening and waterproofing where durability is essential, and the above characteristics must be provided. Four categories can be recognized: acrylic, phenolic, aminoplastic, and polyurethane as indicated in table 3. Chrome lignosulfonates are not discussed, because of the environmental damage caused by the highly toxic and dermatitic components.

Table 3. Uses and applications of resins.

Type of Resin	Nature of Ground	Use/Application
Acrylic	Granular, very fine soils Finely fissured rock	Waterproofing by mass treatment Gas tightening (mines, storage) Strengthening up to 1.5 MPa Strengthening of a granular medium subjected to vibrations
Phenol	Granular, very fine soils	Strengthening
Aminoplastic	Schists and coals	Strengthening (by adherence to materials of organic origin)
Polyurethane	Large voids	Formation of a foam that forms a barrier against running water (using water-reactive resins) Stabilization or localized filling (using two-component resins)

Of these four subclasses, only the two following groups of polyurethanes are usually appropriate for grouting:

- Water-reactive polyurethanes: Liquid resin, often in solution with a solvent or in a plasticizing agent, possibly with added accelerator, reacts with groundwater to provide either a flexible (elastomeric) or rigid foam. Viscosities range from 50 to 100 cP. They may be either:

  - hydrophobic - react with water but repel it after the final (cured) product has been formed or,
  - hydrophillic - react with water but continue to physically absorb it after the chemical reaction has been completed.

- Two component polyurethanes: Two compounds in liquid form react to provide either a rigid foam or an elastic when supplemented with a polyisocyanate and a polyol. Such resins have viscosities from 100 to 1,000 cP and strengths as high as 2 MPa. A thorough description of these grouts was provided by Naudts.[38]

4. Miscellaneous Grouts

These grouts are essentially composed of organic compounds or resins. In addition to waterproofing and strengthening, they also provide very specific qualities such as resistance to erosion or corrosion, and flexibility. Their use may be limited by specific concerns such as toxicity, injection and handling difficulties, and cost. Categories include hot melts, latex, polyesters, epoxies, furanic resins, silicones, and silacsols. Some of these (e.g., polyesters and epoxies) have little or no application for ground treatment. Others such as latex and furanic resins are even more obscure and are very infrequently encountered in practice.

For certain cases in seepage cut off, hot melts can be a particularly viable option. Bitumens are composed of hydrocarbons of very high molecular weights, usually obtained from the residues of petroleum distillation. Bitumen may be viscous to hard at room temperature, and have relatively low viscosity (15 to 100 cP) when hot (say 200 degrees C plus). It is used in particularly challenging water-stopping applications, remains stable with time, and has good chemical resistance. Contemporary optimization principles require simultaneous penetration of the placed bitumen mass by stable particulate grouts to ensure good long-term performance of the system.

Also of considerable potential is the use of silacsols. Silacsols are solution grouts formed by reaction between an activated silica liquor and a calcium-based inorganic reagent. Unlike the sodium silicates discussed above, aqueous solutions of colloidal silica particles disperse in soda, and the silica liquor is a true solution of activated silica. The reaction products are calcium hydrosilicates with a crystalline structure similar to that obtained by the hydration and setting of Portland cement, i.e., a complex of permanently stable crystals. This reaction is not therefore an evolutive gelation involving the formation of macromolecular aggregates, but is a direct reaction on the molecular scale. This concept has been employed in Europe since the mid-1980s with consistent success in fine-medium sands.[39] The grout is stable, permanent, and environmentally compatible. Other important features, relative to silica gels of similar rheological properties, are:

- their far lower permeability;
- their far superior creep behavior of treated sands for grouts of similar strength (2 MPa);
- even if an unusually large pore space is encountered, or a large hydrofracture fissure is created, a permanent durable filling is assured.

2.4 GROUTING EQUIPMENT

Many types of grouting equipment are commercially available and are used routinely for grouting operations of different types and scale. The main components are: grout mixing equipment of a capacity adequate for the job, and that mixes grout to a uniform consistency: a storage tank capable of continuous agitation of the grout to prevent settlement and segregation; a pump capable of precise pressure and volume control; appropriate grout parameter recording equipment; and a system of grout lines with a header for injecting grout into the hole as desired. Suitable packers, gauges, valves, and accessories are also required. A typical layout for a slurry injection application is shown on figure 12.

The grout mixer and agitator need not be of the same volume capacity. Where high grout takes are anticipated, two mixers may be arranged to discharge into the same storage tank. Both the mixer and the agitator should continuously agitate the grout until it is either injected or wasted. For slurry grouts, high speed, high shear colloidal grout mixers are superior to standard slow-speed mechanical mixers because they produce grouts of greater uniformity more quickly. Colloidal mixers are required for mixing and hydrating bentonite. Bentonite is mixed in a separate mixer and must be fully hydrated before being introduced into the grout mixer. Water is metered into the mixers, and the meter should be calibrated in liters and be large enough for easy reading. Different mixing principles and equipment apply for compaction, jet and chemical grouting.

Various types of pumps are used, again depending on the application. The pump should be specified based on the individual job requirements. Either piston pumps or progressive cavity pumps are used for slurry grouts, concrete pumps for compaction grouting (modified as necessary), and custom built equipment is used for chemical, and grouting.

Technique-specific aspects regarding equipment are addressed in chapters 3 through 5. Typical examples of drilling and grouting equipment are shown in figures 13 through 25.

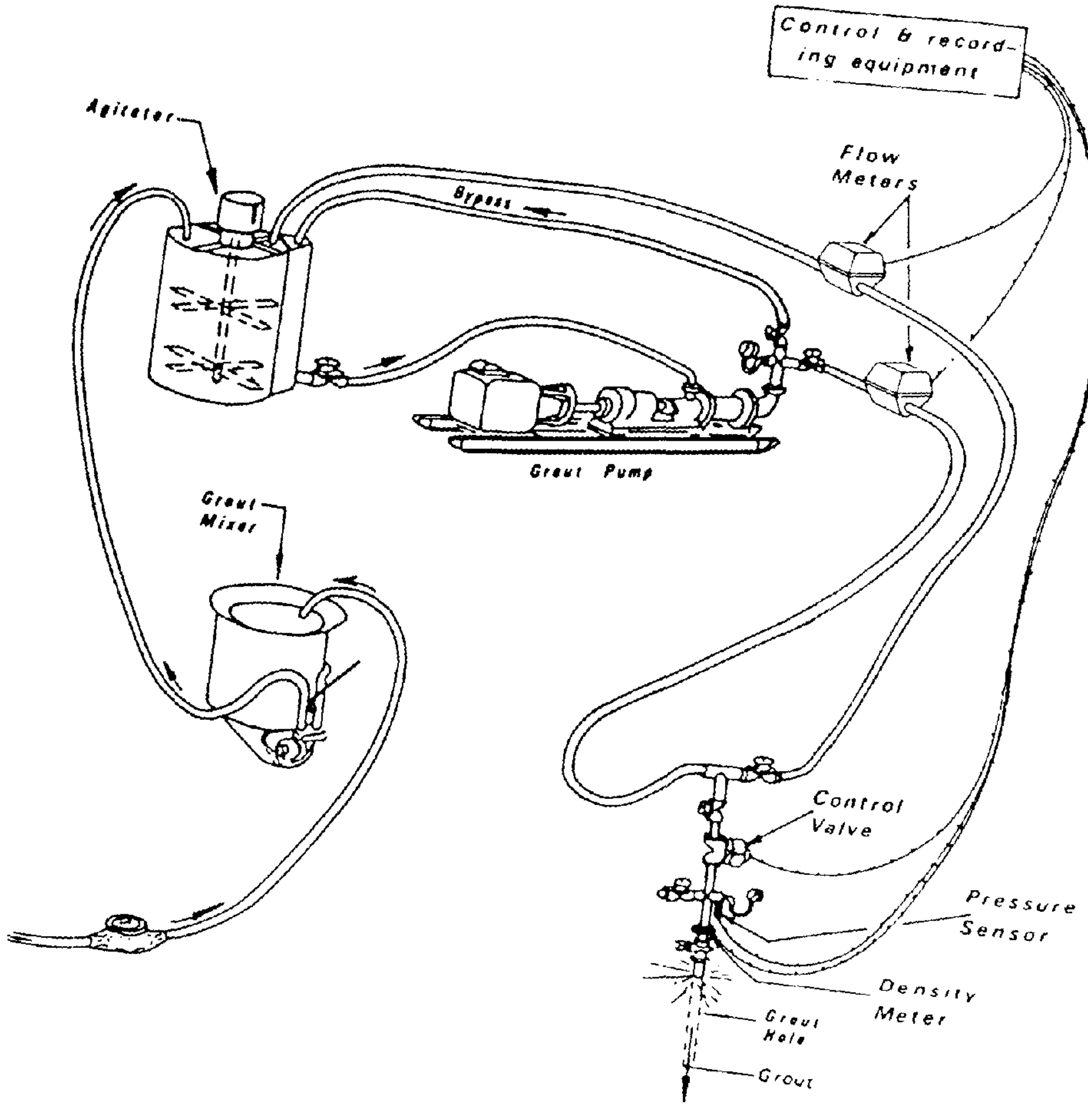


Figure 12. Typical layout for slurry injection.[3]

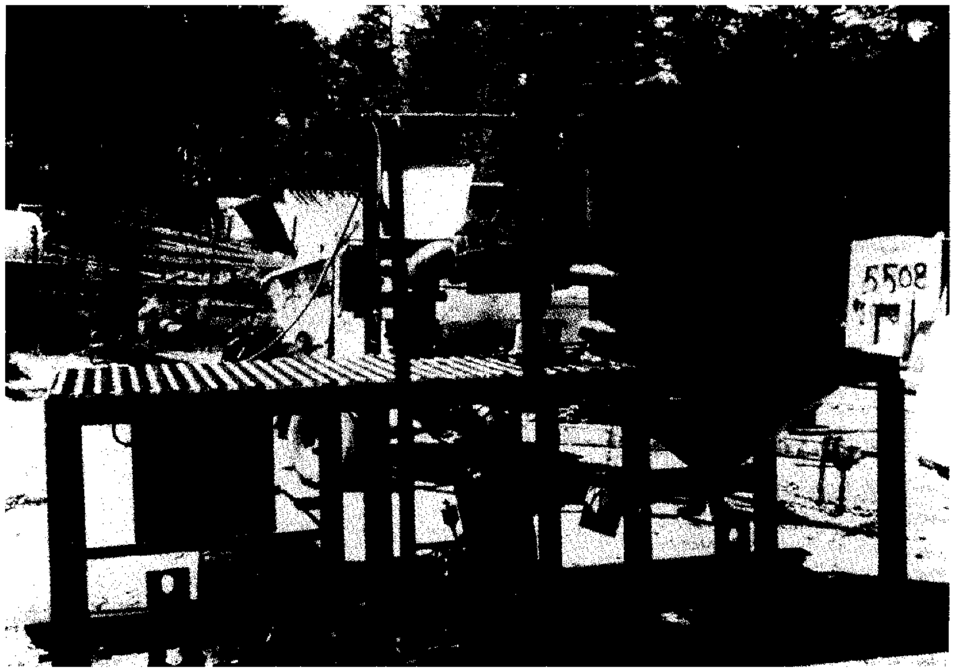


Figure 13. Electric powered high shear slurry grout mixer.

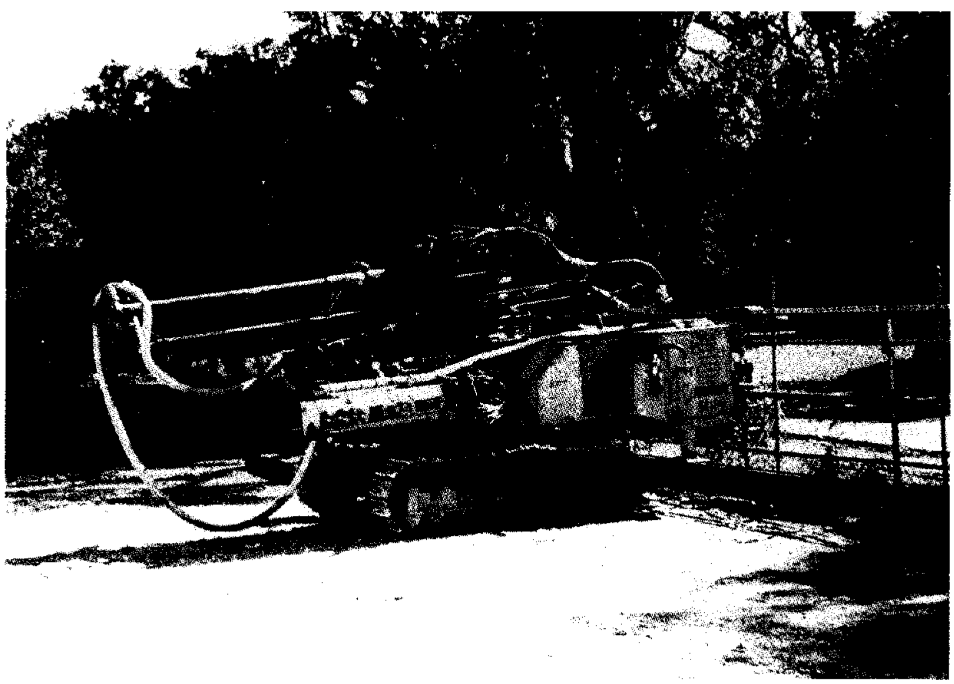


Figure 14. Rotary/rotary percussion diesel hydraulic track drill.

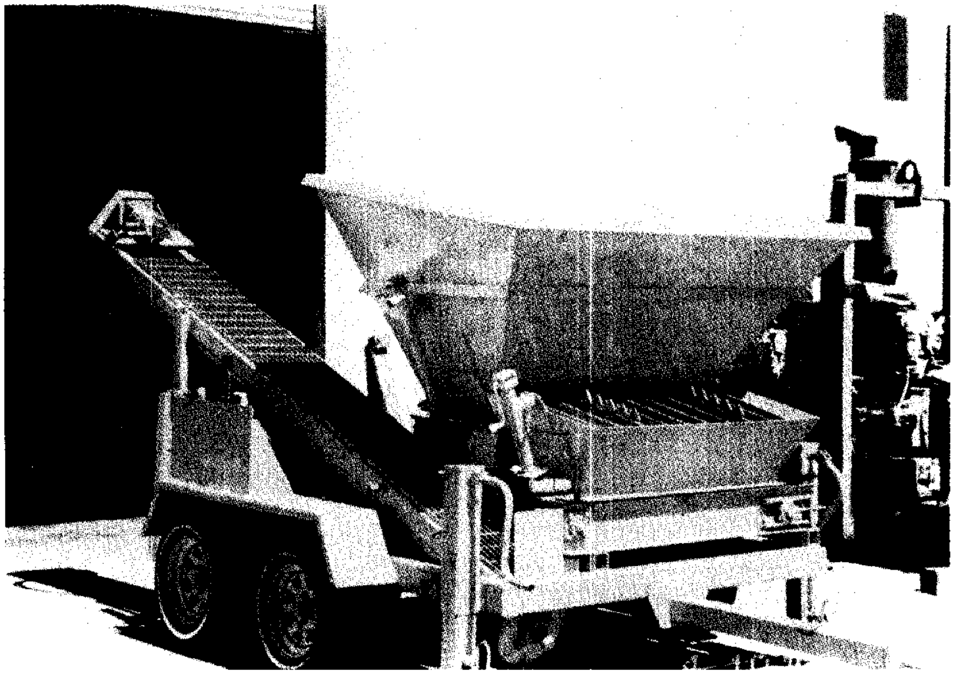


Figure 15. Small compaction grout batcher.

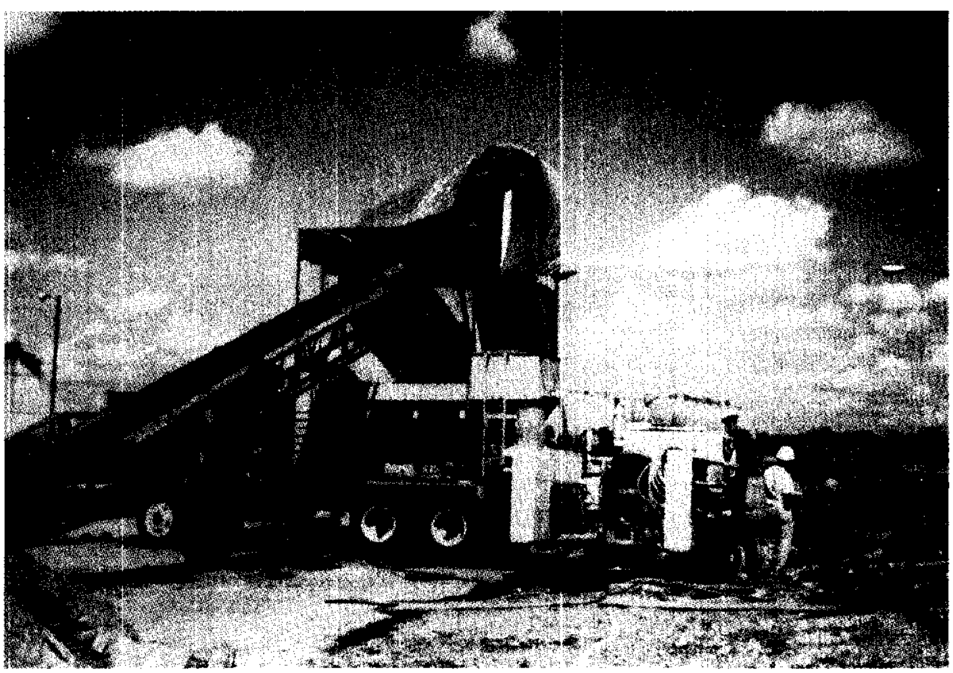


Figure 16. Larger on site grout batching plant for compaction grouting.

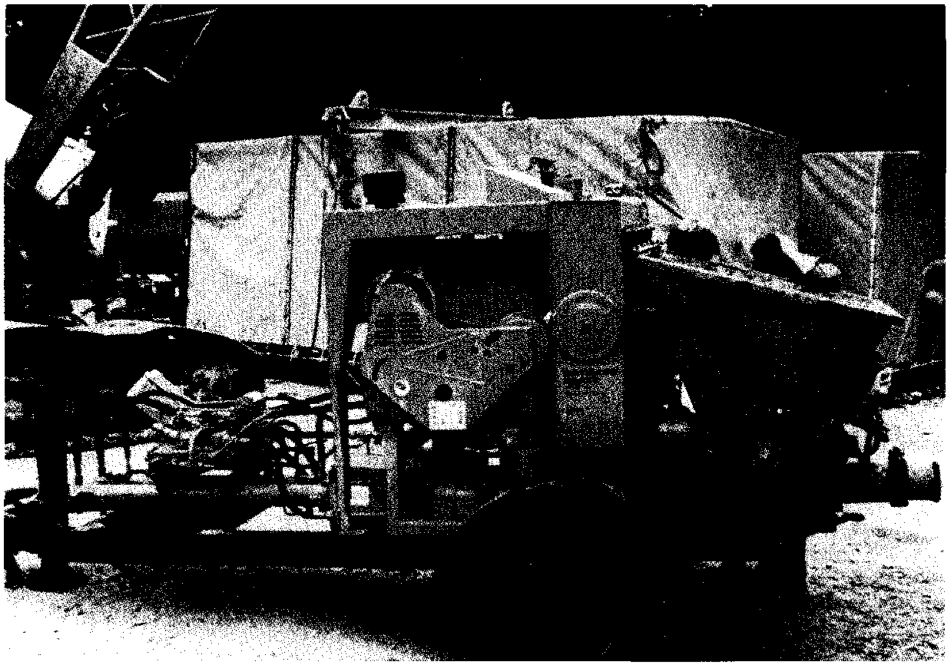


Figure 17. Compaction grout pump.

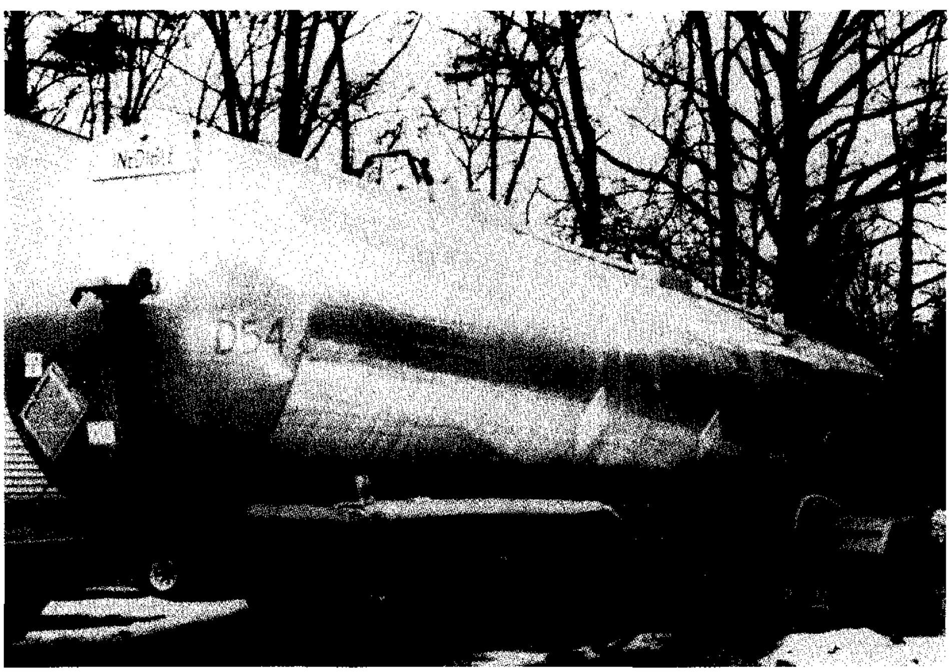


Figure 18. Compartmentalized tanker for raw chemical grout components.

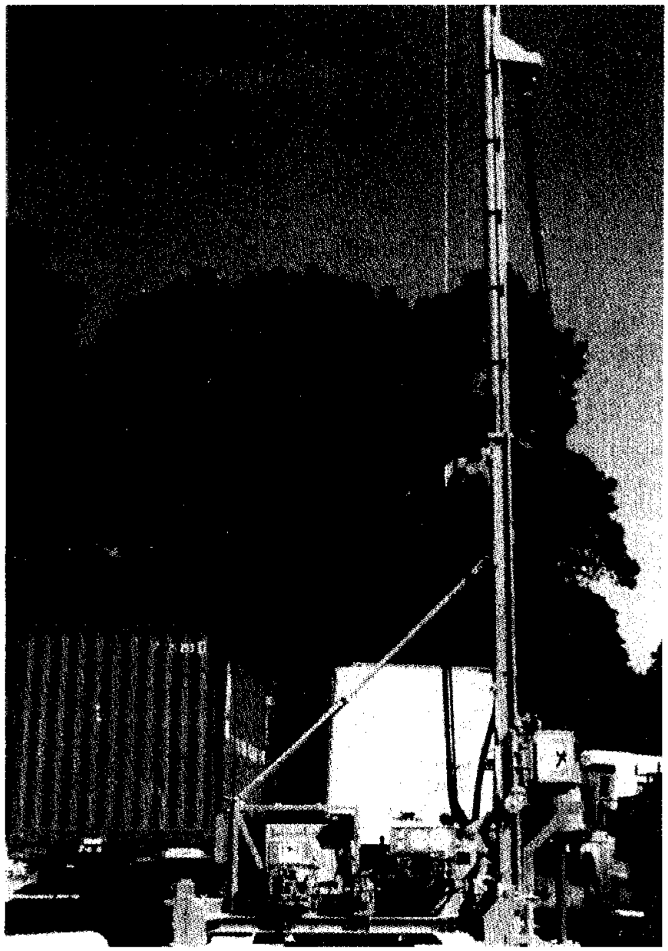


Figure 19. Jet grouting rig.

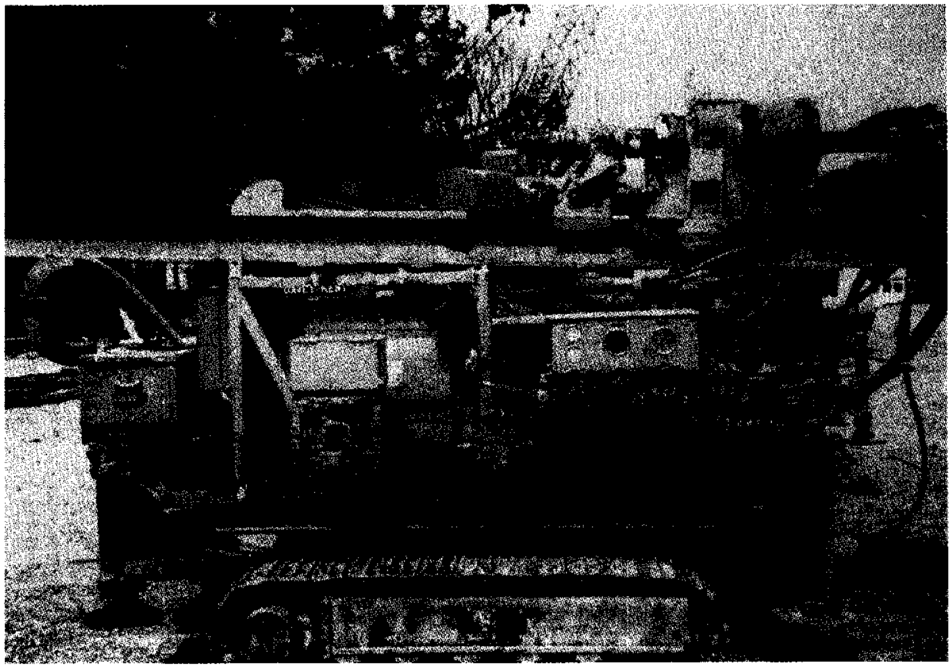


Figure 20. Jet grouting rig (drill mast at rest position).

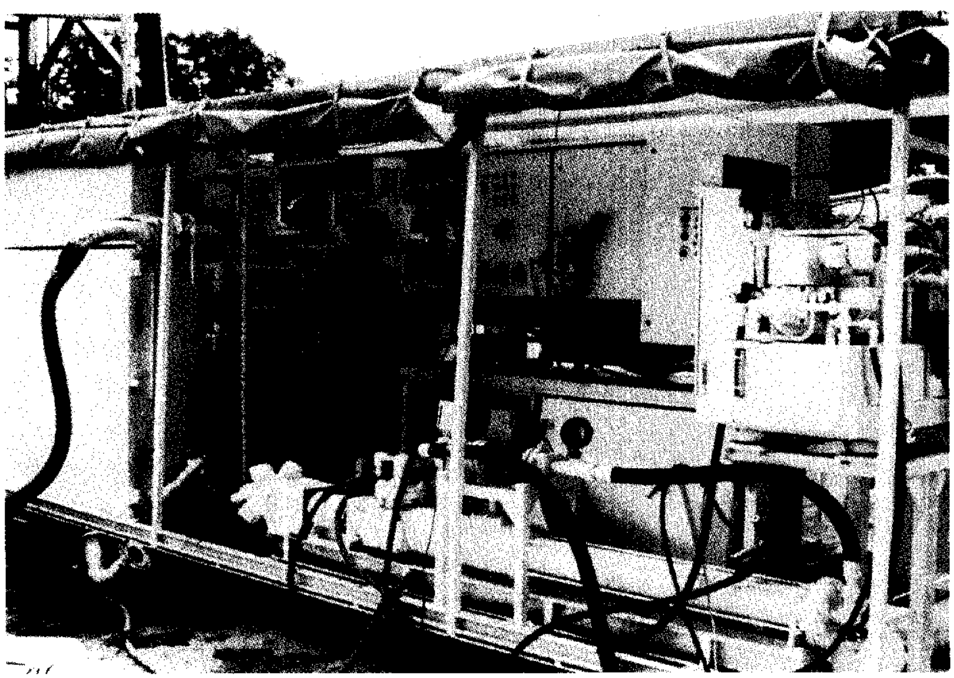


Figure 21. Pumping station for triple fluid jet grouting.

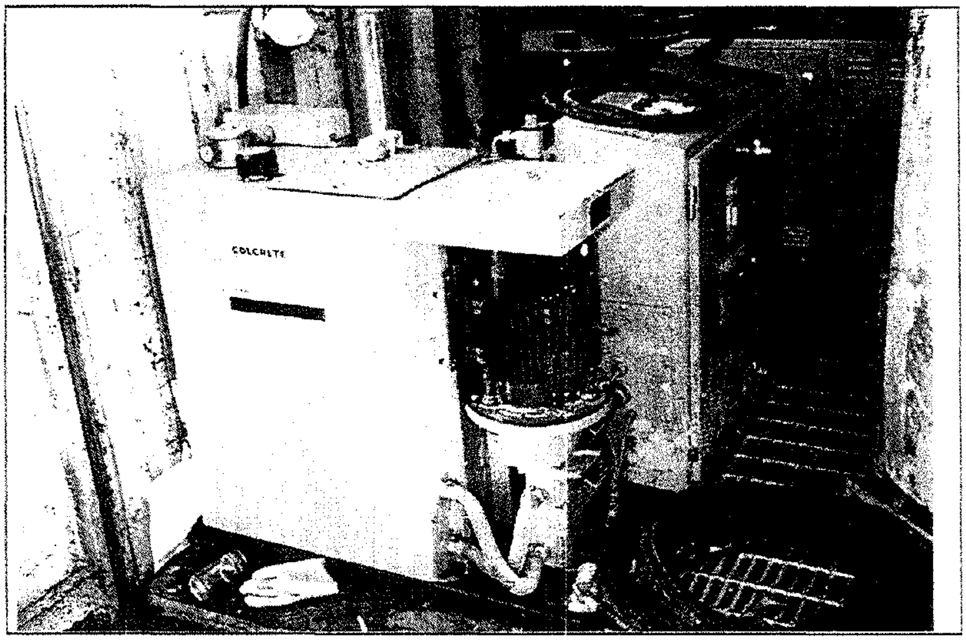


Figure 22. High pressure slurry grout pumping unit.

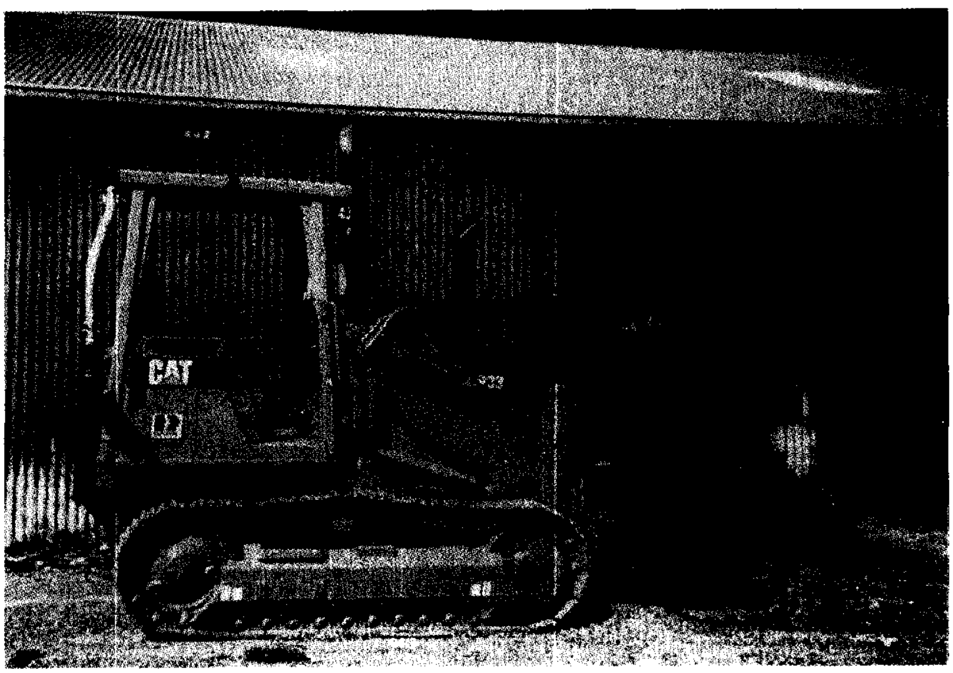


Figure 23. Hydraulic crawler drill for lime injection.

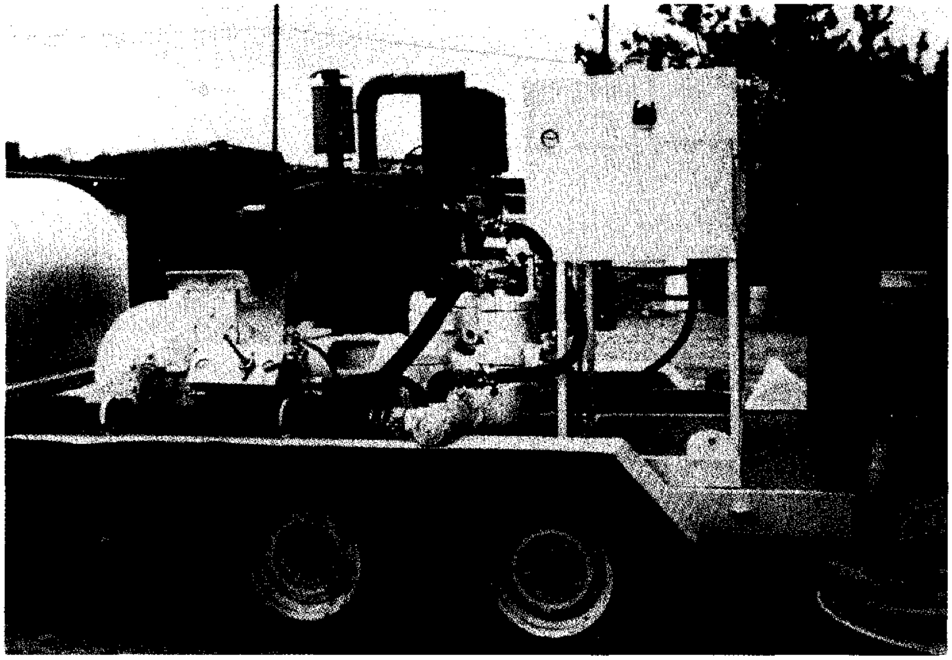


Figure 24. Piston-type pump for lime injection.

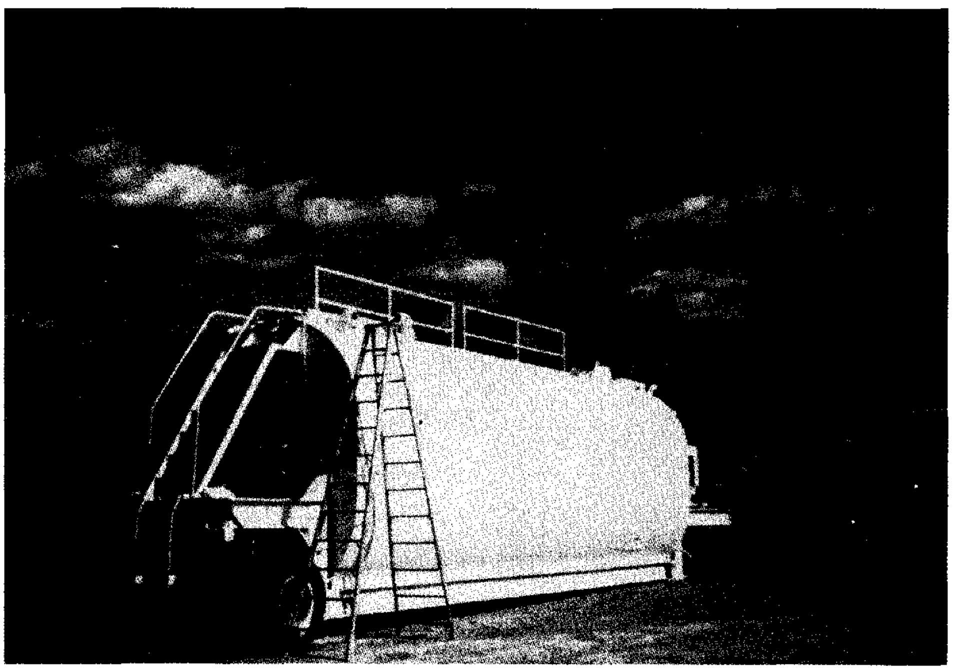


Figure 25. 64.4kL tank with agitator to mix lime injection components.

2.5 QA/QC PRINCIPLES AND VERIFICATION

It is often overlooked that every hole that is drilled on a project is potentially a source of valuable information on the ground at that stage of the project. Whether that potential is realized depends on the accuracy and content of the drill logs which are taken and any tests (e.g., permeability) which are run. This can be done manually or electronically, as previously described. The key is that the data are studied in real time, or very soon thereafter, and that any adjustments or changes to the grouting program can be effected in a timely fashion.

Similarly, the grouting data provides equally valuable information of how the ground is behaving in response to the treatment. Close examination of grout pressure/volume/time records, again manually or electronically recorded and/or displayed, will provide vital insight into the effectiveness of the operation to that point. For example, if a rock grouting operation is progressing well, then the higher order holes will have smaller grout takes, and will need slower rates of injection at equivalent pressures to attain refusal than the primaries.

During grouting, it is essential to frequently and routinely monitor the fluid properties of the materials being injected. Thus, for rock fissure grouting or soil permeation grouting, it is instructive to record the fluidity, the specific gravity, the setting time, and the stability, whereas for compaction grouting only slump testing may be of relevance.

As a further general point, it may be emphasized that the site's geotechnical situation must be "baselined" prior to grouting. This means that the key virgin parameters must be measured (such as density or permeability), depending on the nature of the project. Following the monitored execution of the grouting work, verification testing must be conducted to demonstrate the effectiveness of that work. The nature of the testing must reflect the goals of the project.

Finally, grouting lends itself, and indeed has a great need for, preconstruction test programs. These permit the designer's assumptions and the contractor's methods to be tried, tested and verified prior to the commencement of the production works. This is often overlooked, and, is aimed at enhancing quality and reducing problems, technical and contractual.

CHAPTER 3

SOIL GROUTING

Soil grouting programs are used to achieve a variety of ground treatment objectives and a number of soil grouting techniques are available. Some techniques, like compaction grouting, permeation grouting, and lime-injection grouting, are well established. Others, like jet grouting and soil-fracture grouting are relatively new in the United States. This chapter discusses these techniques and their applications as well as the advantages, disadvantages, and limitations of each. Design considerations, program design and construction, and costs are addressed. Case histories are presented to illustrate specific applications of each technique.

3.1 APPLICATIONS

Soil grouting can be conveniently divided into two major groups of applications:

- Grouting for water control and waterproofing, and
- Structural grouting.

Within each class of treatment, one or more of the grouting techniques may be applicable.

For the purposes of this technical summary, water control is construed to be used in conjunction with new construction, and waterproofing used in conjunction with remedial applications. Applicable techniques are permeation and jet grouting, while compaction grout can be used for both water control and water proofing.

Soil grouting techniques that can be applied for structural grouting are:

- Permeation grouting
- Compaction grouting
- Jet grouting
- Soil fracture grouting
- Lime injection

The major structural applications are:

Densification

The density of all granular soils above and below the ground water table can be improved by various in situ techniques such as dynamic compaction, vibro-compaction, stone columns, and compaction grouting. These are only applicable to new construction. For densification of loose granular soils under existing structures, compaction grouting has proven to be effective.

Raising Settled Structures

Successful raising of settled structures requires a controlled grouting operation. Although slurry grouting has successfully raised slabs and footings, its major disadvantage is the lack of control of the flowable mixes. Both compaction grouting and soil fracture grouting can be precisely controlled for structural settlement remediation.

Settlement Control

Depending on the soil type, cost and potential cause of settlement, permeation, compaction, jet, fracture, and lime injection grouting can be effective in controlling post construction settlement.

Underpinning

A structure is normally underpinned to prevent settlement from occurring due to adjacent, planned construction or when it is proposed to add additional loads to a foundation. Depending on the soil beneath the structure to be underpinned, permeation, compaction, jet and soil fracture grouting can offer alternatives to other underpinning techniques.

Excavation Support

Soldier piles and lagging, sheet piles, and structural diaphragm walls, with or without tiebacks or internal bracing, are the conventional methods of excavation support. However, when structures or utilities can be affected by the installation of these systems, permeation or jet grouting can be viable alternatives.

Soft-Ground Tunneling

Potential settlement is a design consideration on all soft-ground tunneling projects. Permeation, compaction, jet and soil fracture grouting can be effective in preventing or compensating for this type of settlement.

Liquefaction Mitigation

Where structures are built on soils that are determined to be liquefiable, permeation, compaction, and jet grouting are potential methods for liquefaction mitigation.

Water Control

Permeation and jet grouting have proven to be effective in controlling groundwater infiltration in underground construction elements, while existing structures experiencing water infiltration can often be made impervious by permeation, jet grouting and the use of compaction grouts.

3.2 DESIGN CONSIDERATIONS

a. Advantages, Disadvantages and Limitations

Soil grouting is an in-situ treatment and so can usually offer a distinct economic advantage over removal and replacement. Another advantage over removal and replacement techniques is safety. For example, grouting for underpinning requires no excavation beneath structures and thus eliminates the need for personnel to work in high risk areas. Grouting is also generally less disruptive to the surroundings of the work site, and this can be of particular importance in residential areas.

With compaction grouting in finer, saturated soils, the instantaneous pressure exerted can fail to squeeze the pore water pressures out of the fine-grained soils so that densification or consolidation may not be achieved and simple displacement of the soil may occur. Permeation grouting using certain chemical grouts may represent toxicity dangers to groundwater and the underground environment. Low toxicity chemical grouts are sufficiently available now for most purposes and should be specified except for unusual circumstances. Each grouting method (especially jet grouting) can cause ground movement and structural distress. This must be carefully guarded against.

The general limitation of soil grouting is the soil type to be treated. Although the range of soil grouting techniques available encompasses most soil types, individual techniques are limited to specific soils, except for jet grouting, as shown in figure 26.

In addition, the full scope and cost of the required program can seldom be determined accurately during the evaluation or design phase. Further, the effectiveness of some applications cannot be predicted with a great degree of certainty during the design phase. Another limitation is the low level of knowledge on all aspects of grouting by the civil engineering community.

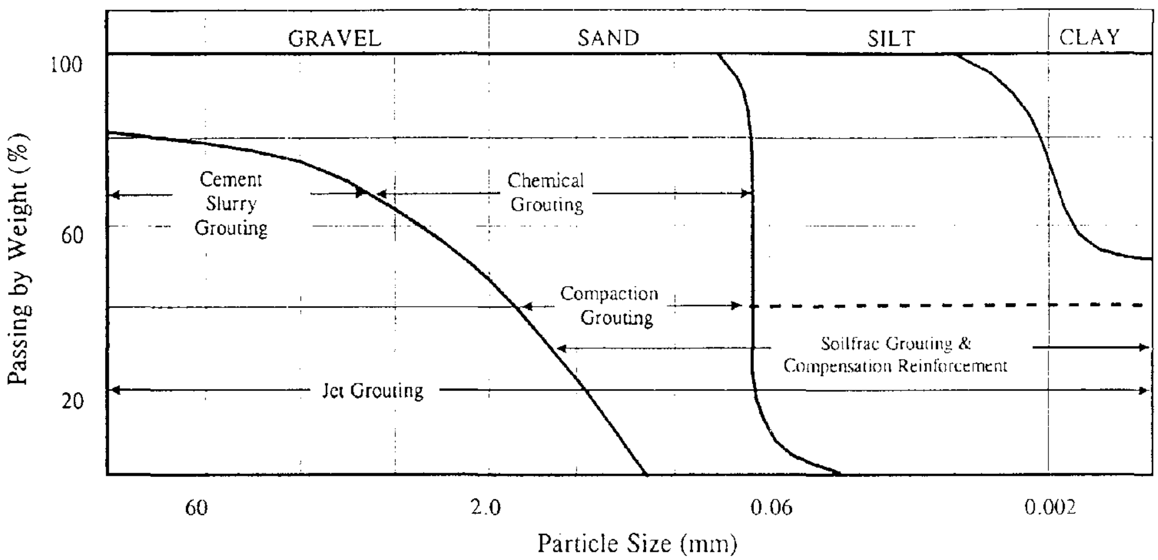


Figure 26. Range of applicability of soil grouting techniques.

b. Feasibility Evaluation

Grouting is typically used to solve construction problems related to geological anomalies or special environmental conditions. Unlike alternate solutions such as deep foundations which bypass the problem soil, a soil grouting solution uses the existing soil, improving it by grouting to make up for the soil's deficiencies.

Grouting of a soil involves the following sequential steps:

- Establishing specific objectives for the grouting program (designer).

- Defining the geometric and geotechnical project conditions (designer).
- Developing an appropriate grouting program design and companion specifications and contract documents (designer).
- Planning the grouting equipment needs and procedural approach (contractor).
- Monitoring and evaluation of the grouting program (designer, contractor).

The flow chart shown in figure 27 illustrates this process more fully.

Pregrouting subsurface investigation programs will normally require more than the usual number of borings, and should include continuous samples and laboratory tests. These tests should include grain size analysis, permeability, pH and other soil index properties. The purpose of the subsurface investigation is to define the limits and characteristics of the geotechnical situation to be solved by the grouting process.

Equally important is the clear identification of the geological subsurface conditions that will control and permit the success of the grouting approach. This includes a thorough knowledge of the stratigraphy, environment, and groundwater.

Stratigraphy

Stratigraphy, including the variations in soil properties, especially permeability in the grouting zone, is an important controlling factor in the design and effectiveness of the grouting process. A well-defined stratigraphic picture can be developed by obtaining continuous soil samples. If split-spoon sampling is being performed, at least two 350 mm-long drive samples should be obtained for every 750 mm of hole instead of one sample per 1.5 meters as is the usual practice. Samples should be retained in their entirety for inspection and classification by a geotechnical engineer. Small, fine-grained lenses should be noted, and grain-size tests should be performed on representative samples of separate micro layers. Considerably more descriptive detail should be shown on a boring log for the grouting specialist than is usually shown on a conventional boring log.

The gradation results should be correlated with the stratigraphy. If the total specimen obtained in a split-spoon test is mixed and used to perform a washed-sieve analysis, the location of silt layers will be missed. The analysis of grain-size curves should be done in conjunction with an understanding of the micro-layering effects present in the soil.

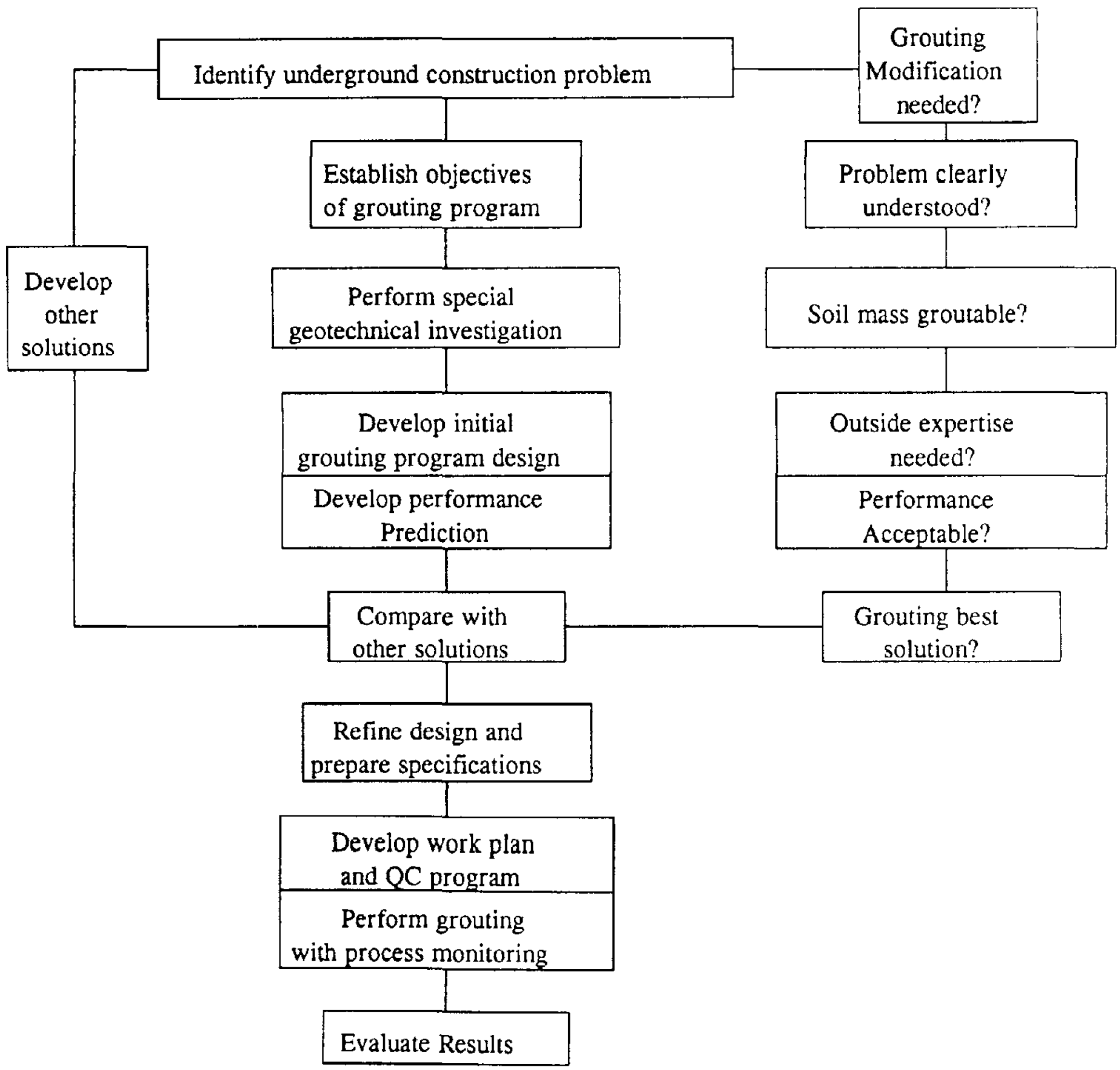


Figure 27. Grouting decision flowchart.

Site History

The overall site environment should be evaluated with respect to the grouting program, particularly how subsurface conditions, both man made and natural, will affect the grouting and how they, in turn, will be affected. For example, if the grouting is to protect the neighborhood from damage during subsurface construction, nearby structures may be able to tolerate only a small amount of deformation. The presence of old shafts, wells, cisterns, etc. within the zone to be grouted can provide a preferred migration path for the grout away from the target soils. Utility trenches backfilled with gravel or sand bedding materials will also provide excellent conduits for grout migration. Old topographic maps can be helpful in piecing together the history of the area.

Grouting technicians and drillers should record every anomaly encountered in the drilling and grouting operations. Such anomalies include a sudden drop in the drill rods, sudden increases or decreases in the ease of drilling, sudden fluctuations in grouting pressures (especially after grouting has been proceeding at a particular stage for some time), and inconsistencies in development of injection pressure with flow rate. These anomalies should be explained and their significance evaluated before conducting any further drilling or grouting.

Consideration should be given to the effect of plugging underground drainage channels and to the additional ground forces that will be created within the grouting zone. Active drain lines and sewers should be monitored to detect any grout infiltration. For sodium silicate grouts, this can be effectively done with recording pH meters fitted with audible alarms that are activated when effluent pH reaches a certain level.

Ground Water

Grouting can be performed in permeable soils either above or below the groundwater surface with about equally successful results, provided both the chemical and hydraulic effects of the groundwater are taken into account.

Samples of the local groundwater should be tested for compatibility with the grouts to be used. Groundwater with high pH can be very destructive to sodium silicate-based grouts, preventing initial gel formation and/or causing grout degradation with time, whereas, soils with very low pH can be very destructive to portland-based cement grouts. However, low pH groundwater conditions can accelerate setting of sodium silicate grouts while preventing the setting of acrylamide or acrylate grouts. The presence of organic materials in the ground or groundwater can also have a dramatic effect on the gel times and quality of chemical and cement grouts. Chemical analysis of groundwater is useful in this respect but should not replace at least one series of grout mixing tests using a groundwater sample in the grout mixture. Additional grout mixing tests should be performed using samples of the actual water source to be used for the job.

During the geotechnical investigation, it is important to establish the directions and rates of any groundwater flow, to distinguish between perched water and groundwater, to establish the presence of any artesian pressures, and to estimate the possible effects the grouting program will have on the groundwater levels.

Once project objectives and geotechnical conditions have been defined, an appropriate grouting program can be developed. Selection of the type of grouting is governed by the desired engineered product and the subsurface conditions.

3.3 PERMEATION GROUTING

Permeation grouting either with particulate or chemical grouts is utilized to give cohesion and/or to reduce the permeability of the soils in that changing the structure or volume of the virgin soil mass. The type of grout utilized will depend on the grain size of the in-situ soil and the results derived from the grouting operation as previously illustrated in figure 3.

a. Applications

As previously described, permeation grouting is an option in appropriate soils for the following applications:

- Waterproofing, typically for remedial purposes such as subway tunnels.
- Settlement control, underpinning and excavation support of granular soils during excavation.
- Soft ground tunnelling to increase cohesion in as shown on figure 28.
- Liquefaction retrofit mitigation by increasing density and displacing pore water.

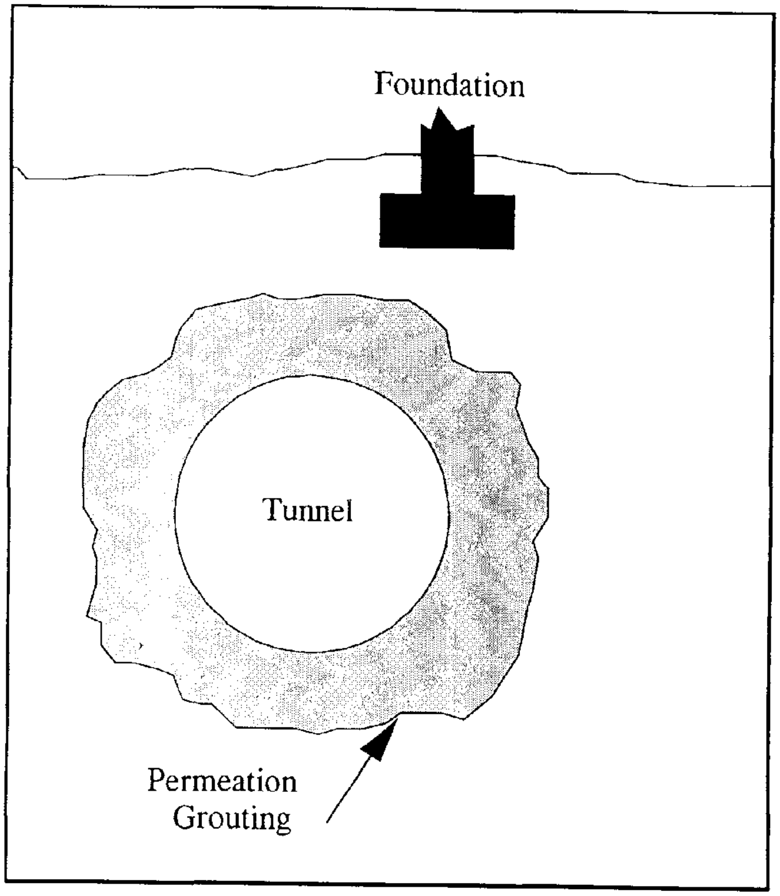


Figure 28. Tunnel excavation support using permeation grouting.

b. Design Considerations

One of the fundamental questions that must be asked when permeation grouting is first considered is whether the ground involved is groutable. A "groutable" soil is one that will, under practical pressure limitations, accept permeation by a given grout at a sufficient flow rate to make the project economically feasible. The permeability of sands may vary as much as three or four orders of magnitude, from 1 cm/sec for medium-grained clean sands to as low as 10^{-5} cm/sec for sand containing 25 percent or more silts and clays. For very low permeability sands, the injection rate at permissible pressures may be so slow that grouting becomes unfeasible. *Thus, permeation grouting is recommended only in predominantly sandy materials with less than 15 percent silts and clays.*

Practical injection rates range from about 2 to 20 liters/min, but they can be as low as 1 liter/min and as high as 40 liters/min. Injection rates higher than 40 liters/min would indicate that the grout is being accepted by the ground very easily, and that steps should be taken (e.g., change of rheology or grain size) to limit the flow. Injection rates slower than 1 liter/min become impractical, since the volume of grout placed per day at this rate, even with a multiple-hole injection system, is very low. In addition, low flow rates may require unacceptably long gel or stiffening times to obtain adequate flow time within the soil for practical grout port spacing.

Initial soil permeability is the primary guide to establishing the groutability of a soil mass. Soils having permeabilities in the range of 10^{-1} cm/sec to 10^{-3} cm/sec are readily groutable. Soils showing permeabilities in the range of 10^{-3} cm/sec to 10^{-4} cm/sec are marginally groutable. When the permeability is from 10^{-4} cm/sec to 10^{-5} cm/sec, the soil is usually ungroutable from a practical point of view.

A preliminary determination of soil permeability, and thus groutability, can be made by measuring the percentage of fines passing a 74 μ sieve. *Soils are initially classified as readily groutable if they have less than 12 percent fines, moderately groutable for 12 to 15 percent fines, and only marginally groutable for 15 percent to 20 percent fines.* Sands are usually considered ungroutable if they have more than about 20 percent fines. Figure 29 shows typical grain-size ranges for soils amenable to permeation by typical silicate grouts.

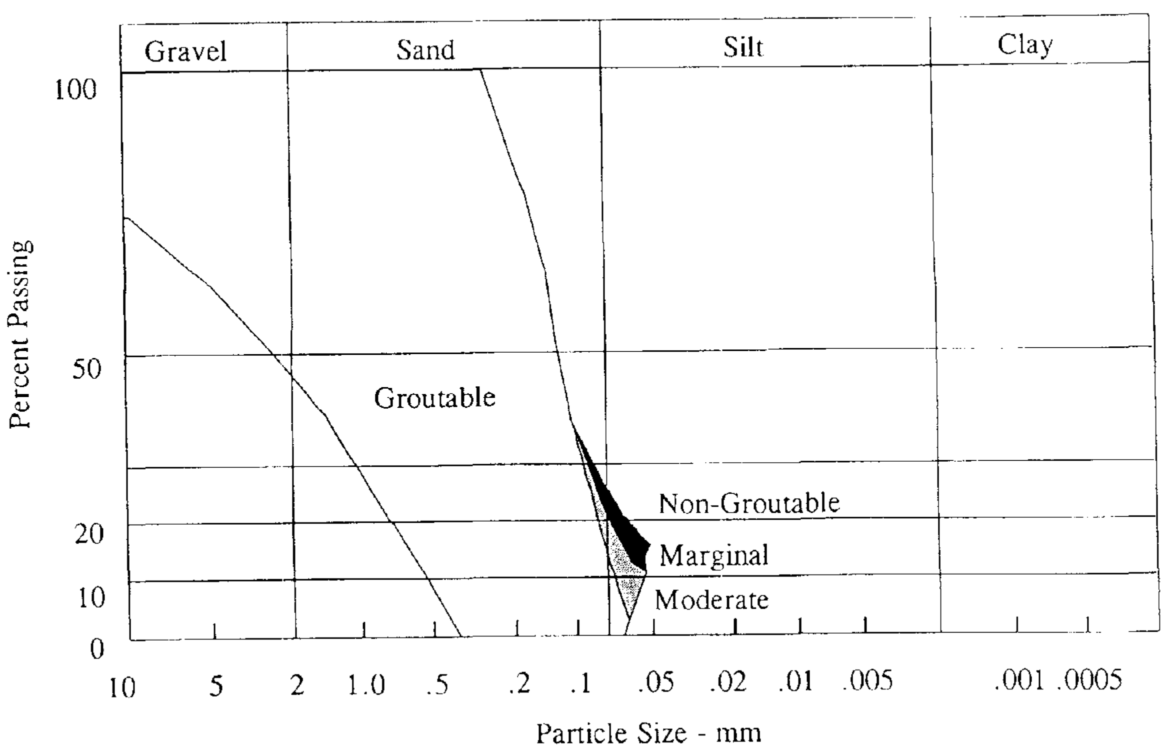


Figure 29. Typical grain-size curves for soils which can be permeated.

A more absolute groutability classification can be based on the results of laboratory and especially field injection tests. The composition of the fines also appears to be important in that the clay content of fines is more effective than the silt content in reducing the groutability of sandy soils. Where many soil specimens are to be evaluated for the amount of material fines passing a 74μ sieve, it may save considerable time to perform Sand Equivalent Tests (ASTM D 2429) and correlate the results with a few Fines Tests (ASTM D 1140) and Laboratory Permeability Tests. As a further guide to permeation potential by particulate grouts, Mitchell proposed that when $D_{15\ soil}/D_{85\ grout} < 11$ permeation is impossible, but easy at ratios >24. Alternatively for $D_{10\ soil}/D_{95\ grout} < 6$, permeation is impossible, but easy at ratios >11.[54]

These proposals were supported by the work of De Paoli et al. who did confirm that the limits of penetrability could be enhanced by using correctly balanced (i.e. stable, low cohesion) grouts.[55]

c. Design of Grouting Program

The term "structural permeation grouting" is applied where the objective of the grouting is to improve the strength and/or rigidity of the groutable soils to prevent ground collapse, reduce otherwise unacceptable ground movement during construction, improve bearing capacity, etc. The term "waterproof grouting" is used to describe permeation grouting aimed primarily at stopping the flow of water, which otherwise would provoke ground movements or the flow of unacceptably large amounts of water into a construction area, or both. Although many grouts (including properly formulated particulate grouts) can be considered to be permanent, i.e., have a service life in excess of 20 years under normal conditions, most structural chemical grouting is required for only a few days to several months. Sodium silicate grouts cannot be regarded as permanent.[40, 38]

The design objective for structural grouting is often to give non-cohesive ground (no strength under unconfined conditions) sufficient cohesion to prevent the beginning of collapses or soil "runs" into excavations, tunnels or shafts. Grout underpinning is another application of structural grouting, wherein granular foundation support soils are strengthened so as to permit excavation adjacent to footings. In these cases, the soil strength lost by the reduction in confining stresses is replaced by the cohesion imparted to the soil by the grout.

The early establishment of clear, quantitative objectives to be achieved by a permeation grouting program is a basic prerequisite to good design and satisfactory, economical performance.

Program Design

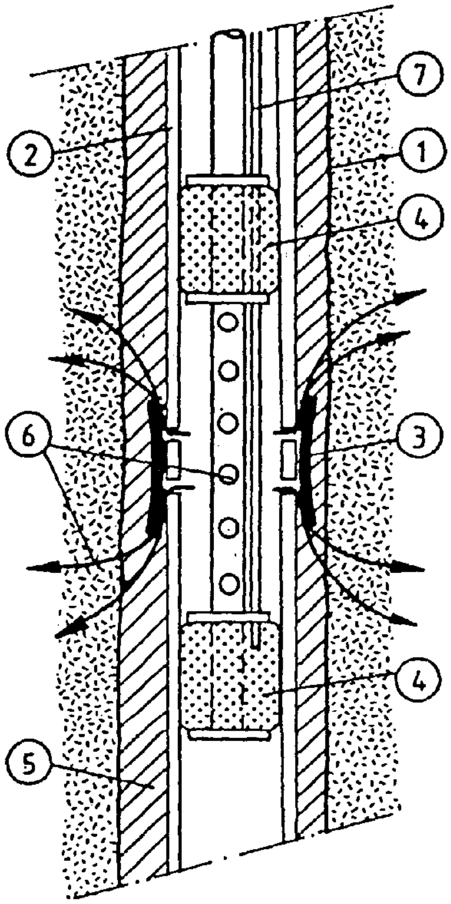
These are well defined equations which can be applied to accurately design the spacing of grout holes for permeation.[32] These require knowledge of the granulometry of the soil and the rheology of the grout, as well as the anticipated flow rates and limiting pressures. However, for preliminary cost and feasibility evaluations, the following guidelines may be considered:

Spacing: Spacing of grout pipes may vary from 0.5 m to 1.5 m for waterproofing and from 1.0 m to 1.6 m for structural applications.

Equipment: Sleeve-port grout pipes, also called "tubes-a-manchette" as shown on figure 30 should be used on all permeation grouting as opposed to basic, end-of-casing injection of materials as the casing is gradually withdrawn. They allow for a well planned primary-secondary grout program horizontally and vertically. The system consists of a three to six centimeter diameter plastic pipe that has grout holes drilled through the pipe wall at distinct vertical locations. The grout holes are covered with a rubber sleeve which acts as one-way check valve. The grout pipe is installed in an oversize borehole and the annular space between the pipe and the borehole wall is filled with a brittle but weak cement-bentonite grout. This grout sheath is fractured when the sleeve is expanded during grouting from inside the pipe utilizing a double packer. The sleeve-port can be injected in any sequence (although always from the bottom up in any are hole) and maybe re-injected if desired. These ports can also be tested with water to check the permeability of the soil before or after grouting. The grout pipes can also be used to run cross hole shear wave velocity tests before and after grouting.

The permeability of the soil in both horizontal and vertical directions should be evaluated in order to predict the relative shape of grout bulbs. It is a common experience to observe elliptically shaped, isolated, grout bulbs with height to diameter aspect of about 0.80 because the horizontal permeability is greater than the vertical permeability. Soil anisotropy will affect the selection of grout pipe spacing and grout port spacing, as well as the sequence in which primary and secondary holes are grouted.

If unexpected, ungroutable lenses occur periodically throughout the design grouting zone, they will control and greatly influence the direction of migration of grout from the grout pipe location. If major ungroutable pockets are encountered, their presence, especially if unanticipated, will significantly influence the effectiveness of the grouting program.



1 Wall of borehole
2 Tube-a-manchette
3 Opened valve (manchette)
4 Double packer
5 Sleeve grout
6 Grout pipe and grout flow
7 Pipe to inflate the packer

Figure 30. Mode of operation of tube-a-manchette.[7]

The original stratigraphic profile should be confirmed during the borings conducted for placement of grout pipes. Since wash or blow samples are generally obtained during grout pipe drilling, and the drillers may not be experienced in geologic drilling, it is important that they report all observed changes in response to the drilling, including changes in drilling rates and wash water.

Grout Quantities: In order to calculate the volume of grout needed to treat a given soil volume, one must have a fairly accurate estimate of the porosity of the soils to be grouted. Typical groutable soils have porosities of 0.25 to 0.45 and it is conventional to assume that the total void space will be filled with grout. For a porosity of 0.35, 350 liters of grout will be required for every cubic meter of soil treated. Depending on the grain size curve analysis, it may be possible to treat larger pores first with an appropriate, economical, particulate grout. So, in this case the 35% porosity may be split 10% particulate, and 25% chemical, for example. Because a major cost of permeation grouting is the chemicals themselves, the porosity has important cost consequences. Estimates of soil porosity are often obtained from previous correlations with Standard Penetration Test "N" values. Where relatively undisturbed samples are obtained, unit weight and specific gravity test results will provide a better estimate of soil porosity for use in grout volume calculations. Equally, permeability tests conducted prior to grouting will give a good indication of the amenability of the soil to different types of ground.

Grout Selection: Permeation grout components for structural grouting are addressed in chapter 2. Waterproof grouting may be used for water control with a grout material having similar properties as outlined in chapter 2 under structural grouting. Other materials occasionally used for water control are acrylates and polyurethanes, although these are used under exceptional conditions only. Selection of grout should take into consideration water flow rates, gel times, and durability. The required residual permeability of the grouted mass will also affect grout selection.

d. Construction Equipment [13]

All chemical grouting equipment should be of a type, capacity, and mechanical capability suitable for doing the work. Equipment for use with particulate grouts is described in chapter 4.

Pumps

The chemical grout plant is usually of the continuous mixing type and should be capable of supplying, proportioning, mixing, and pumping the grout with a set time between 5 and 50 minutes. Batch-type systems may also be used in appropriate conditions. Each main pump should be equipped with sensors to record pressure and rate of injection, as a minimum. The meters should be constructed of materials that are non-corrodible for the intended products and should operate independently of the viscosity of the metered fluid. The pumping unit should be capable of varying the rate of pumping while maintaining the component ratios constant.

Piping and Accessories

The pumping unit for chemical grouting using the proportioning system should be equipped with piping and/or hoses of adequate capacity to carry the base grout and reactant solutions separately to the point of mixing. The hoses should come together in a "Y" fitting containing check valves to prevent backflow. The "Y" fitting should be followed by a suitable baffling chamber. A sampling valve should be placed beyond the point of mixing and the baffling chamber and should be easily accessible for sampling mixed grout. A water flushing connection or valve should be placed behind the "Y" to facilitate flushing the grout from the mixing hose and baffle between grouting sessions. Distribution of proportioned grout, under pressure, to the grouting locations should be monitored by separate, automatic recording, flow rate indicators and gages.

Chemical Tanks

Chemicals should be stored in metal tanks, suitably protected from accidental discharge by valving and other necessary means. Tank capacity should be sufficient to supply at least one day's worth of grouting materials so as not to interrupt the work in the event of chemical delivery delays.

e. Cost

To determine a preliminary estimate for permeation grouting quantities, the volume of sand (cubic meters) to be impregnated is multiplied by a projected 30 percent grout volume factor. Convert to liters by multiplying by 1000 to obtain the anticipated liters of grout to be used. For a project where more than 200,000 liters of sodium silicate grout are anticipated, a cost of $0.65 per liter in place can be used for estimating purposes.

A mobilization/demobilization rate ranging from $10,000 to $50,000 and a cost of providing and installing the sleeve port grout pipes starting at $65.00 per linear meter should be added to the estimate. These budget prices accommodate the cost of any particulate grouting undertaken.

f. Case Histories

Case 1 - Pre-Grouting for Tunnel Construction; L.A. Subway Section A130[41]

Section A130 of the Los Angeles Metro Rail system is a 225 m long, twin tunnel that runs beneath 10 lanes of the Los Angeles Freeway and also crosses beneath a busy intersection. Pre-construction subsurface investigations had revealed 2 to 5 m of loose to medium dense silty sand

and sand fill underlain by an alluvial deposit of discontinuously interbedded sand and gravels to a depth of 13 m. Because of the potentially flowable nature of the clean, granular soils, the narrow 1.8 m pillar between the 6.4 m dia tunnels (figure 31) and the shallow ground cover of 5.8 m at the Freeway and 3.7 m at the intersection, permeation grouting was selected to prevent run-ins and control settlement during tunneling beneath these critical traffic routes to downtown Los Angeles.

A section of the tunnel that crossed beneath a vacant lot, and hence did not present settlement concerns, was not grouted. Full-face grouting of the twin-tunnel bores was specified for the zone beneath the Freeway and at the intersection (figure 32).

Along the Freeway, where traffic disruption was deemed unacceptable, horizontal drilling and grouting was required, with sleeve port grout pipe lengths ranging from 79 to 97 m. At the intersection, where traffic could be re-routed, a surface drilling and grouting program was implemented. Here grout pipes extended to a depth of 12.2 m below the road surface.

Extensive field monitoring, using specially designed computerized equipment, allowed accurate recording of the position of each sleeve port grout pipe and determination of the correct grout quantities. Over 7.5 million liters of Geloc-4 sodium silicate grout was required for the project, making this the largest chemical grouting project in the United States up to 1990.

Comparison of pre-grouting and post-grouting test borings showed a significant increase in blow counts. Surveyed surface settlements in the grouted intersection area, where the ground cover was thinnest, were less than 10 mm, and there was no settlement detectable in the freeway area.

Four months after the completion of the grouting program, a fire occurred in the tunnel, destroying tunnel lagging for the full 225 m length. While the grouted sections suffered only local spalling of the grout, the ungrouted section suffered complete structural failure, illustrating the success of the structural grouting program (figure 33).

Case 2 - Post-Grouting for Water Control of Underground Structures; Baltimore Subway

The Baltimore subway system consists of concrete and steel lined tunnels approximately 19 m below ground surface. Seepage of groundwater through shrinkage joints and cracks had created a potential safety hazard in the subway station area and problems with tunnel track maintenance. Working under a maintenance contract and on an as needed basis, the grouting contractor sealed the leaks by the injection of polyurethane grout and low viscosity acrylate grout into the joints and cracks.

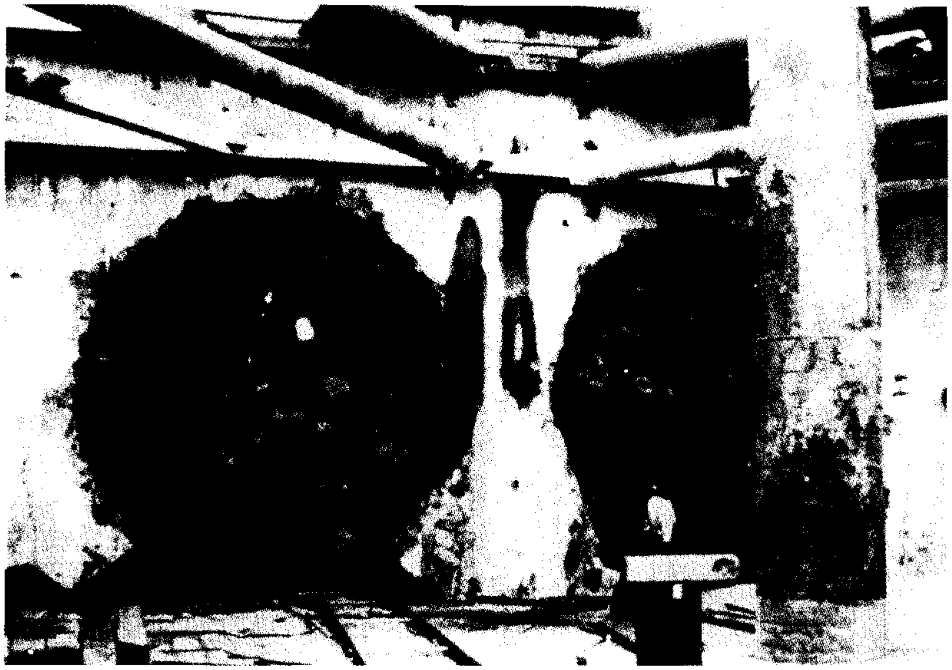


Figure 31. Los Angeles subway twin tunnel

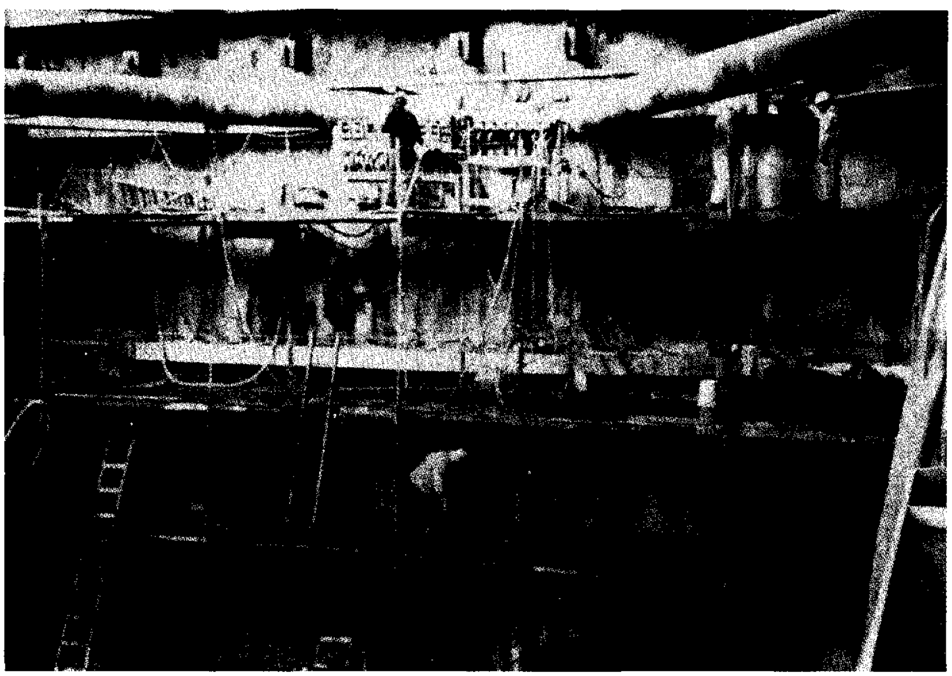


Figure 32. Full-face grout encapsulation in Los Angeles subway.

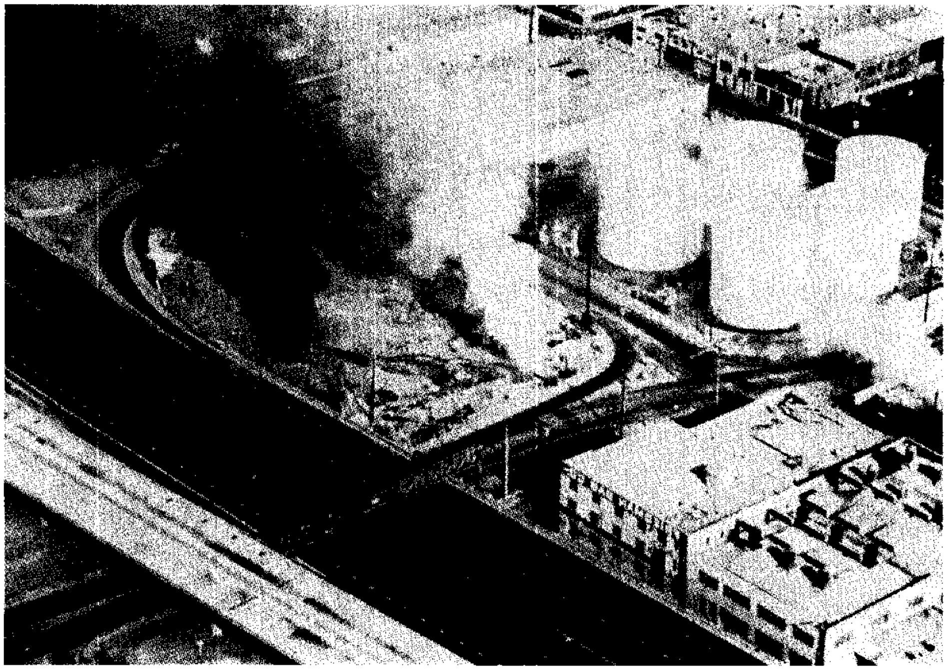


Figure 33. Collapsed tunnel due to Los Angeles subway fire.

Grouting was not permitted while the Metro rail traffic operations was in operation, therefore, all tunnel grouting was carried out between 9:00 pm and 4:00 am. In the station, where revenue producing traffic would not be affected, grouting was accomplished during normal working hours. Since all equipment had to be removed from the work area at the end of each shift, portable, hand-held rotary percussion drills and hand-operated piston type grout pumps were used.

In both the concrete and the steel-lined sections, nominal 22 mm diameter holes were drilled into the leakage zones. In the concrete liner, holes were drilled on an angle to intercept the leaks approximately 150 mm back in the wall. In the steel liner, grout holes were drilled in the vicinity of the leaks to intercept the sources behind the plate.

Two types of chemical grout were used, their use at a particular location being dependent on the amount of leakage and/or size of the opening. Due to its ability to set in flowing water, polyurethane grout (TACHS) was used to seal larger cracks and construction joints exhibiting free-flowing water. Acrylate grout (AC-400) was used to seal shrinkage cracks and tighter, hairline cracks with water.

Maintenance contracts as described above have proved to be very successful in solving water infiltration problems in subway tunnels without disrupting normal operations.

3.4 COMPACTION GROUTING

This process consists of the injection of low slump, low mobility grout into loose or loosened soils of appropriate granulometry.

a. Applications

As noted, compaction grouting can be used in a wide variety of applications including soil densification (for static and seismic enhancement), raising of surficial structures, settlement control over tunnels (figure 34) or sinkholes, and for structural underpinning. Compaction grout can also be used to seal off major water ingresses through open channel systems.

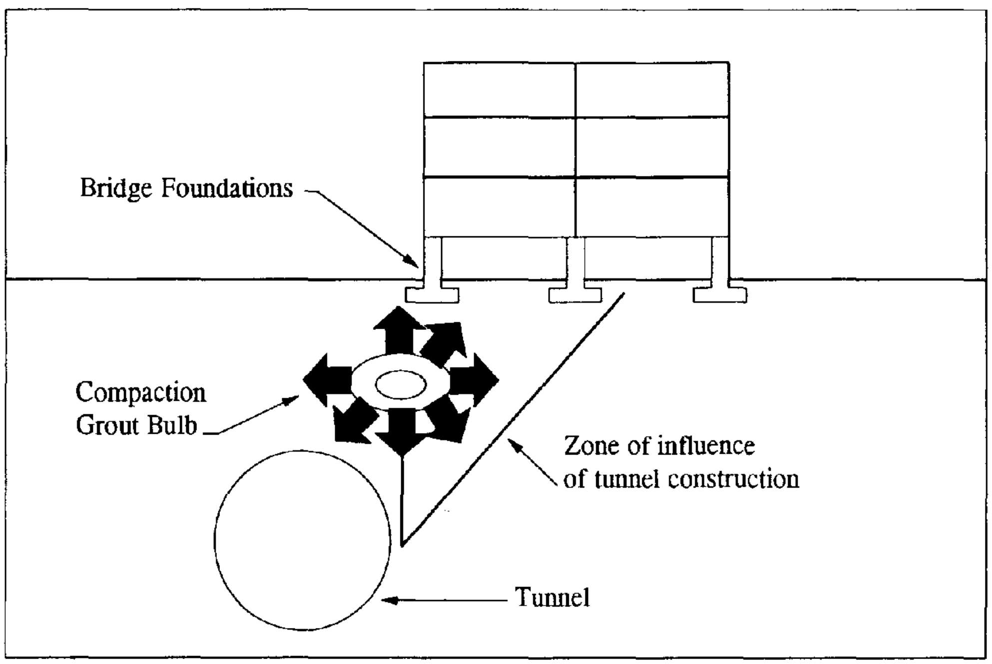


Figure 34. Prevention of tunnel induced settlements using compaction grouting.

b. Design Considerations

Figure 26 indicates the range of soils where densification by compaction grouting may be expected to be effective, i.e. in all relatively free draining soils including gravels, sands and coarse silts. In fine grained soils, pore pressures may not be able to dissipate and improvement may not be economically achievable. Grout mix design is also critical, in that the grout must have high internal friction, to ensure that the bulbs preserves their "spheroidal" shape in the soil. Otherwise, fracturing and lensing will occur, leading to ineffective densification.[56]

For compaction grouts used for void fill, or for water cut off, different rheological properties may be preferable, eg. a slightly higher acceptable slump (up to 100 mm), or the use of polypropylene fibers in the mix. When the mix has to be pumped over long distances (i.e. over 100 m), slump measurements at the pump may be higher (say up to 150 mm).

As in all specialty geotechnical processes, the input of a specialty contractor's should be sought in the development of a well conceived test program. This is especially valid when the purpose of the compaction grouting is to elevate a settled structure, or to compensate for ground loosening under the foundations of an existing structure adjacent to active soft ground tunnelling. There are no mathematical equations available to accurately design grout hole spacing, rates of injection, limiting volumes and so on, as is the case with permeation grouting. There is however, a great deal of project experience and a large number of successful case histories, well documented to guide project implementation.

Regarding site assessment, conventional measurements such as SPT, CPT are typically used. For sinkhole remediation or flow sealing, piezometric data and a variety of geophysical techniques eg. Ground Penetrating Radar, can provide valuable data.

In general, compaction grouting is effective in loose granular soils, loose unsaturated fine grained soils, collapsible soils, and in void filling.

c. Design of Grouting Programs

Spacing: For compaction grouting for densification or redensification, grout pipes are normally installed at 2.4 to 4.6 m intervals for tunneling projects, 2.0 to 5.0 m intervals for site improvement, and 1.0 to 3.0 m for remedial work on existing structures in the area being grouted. Primary holes for use in locating and sealing sinkholes or channel flows will be spaced in relationship to the nature of the problem but may be in the range of 3 to 9 m. In such instances, tertiary holes are usually required to ensure satisfactory performance. The pipe diameter should be at least 76 mm in order to adequately handle the specified low slump material without plugging.

Grout Quantities: Compaction grout quantities will depend on the soil type, its existing density and the density required, or on the size of the void to be filled. For most densification projects, the volume of compaction grout will range between 3 percent and 12 percent of the volume of soil being treated, whereas for void filling, individual stages may consume 10's of cubic meters of grout.

Grout Selection: Portland Type I or II cement is normally used. Fine aggregate is usually a sandy loam with a fines content of not less than 10 percent and not more than 25 percent. Natural fines may be supplemented with fly ash, bentonite, or aggregate washings. Proportions of the mixture are approximately three to six sacks of cement per bulk cubic meter of silty sand, and water as required to achieve a pumpable mix with not more than a 25 mm slump as measured at the header. Depending on the application, other additives to the mix may include gravel, coarse sand, fibers or antiwashant agents.

d. Construction Equipment

Mixers and Pumps

The low mobility grout will require different mixing, pumping, and delivery equipment than more fluid slurry grouts. *"Compaction Grouting - the First Thirty Years"* lists the requirements for the mixers, pumps, and hoses.[20A] Specialized contractors and some grout equipment suppliers have developed their own equipment and are continuing to update this equipment based on their own, on-the-job experiences and requirements.

The grout plant should be designed to handle the specified materials for this type of work. The mixer should be of the pug mixer type to ensure complete uniform mixing of the materials used and should be of sufficient capacity to continuously provide the pumping unit with mixed grout at its normal pumping rate. The pumping unit should be capable of continuously delivering the specified grout materials at appropriate rates and pressures to the grout pipe head. Under certain conditions, it may be possible to use readimix material delivered in mixer trucks to the pumping location. Each truck's load must be carefully tested to ensure compliance with the slump criterion. The inspector must be prepared to reject truck loads which exceed criteria upon delivery, or at any time during the pumping operation rom that batch.

Grout Pipes

Grout pipes should be steel casing of adequate strength to maintain the hole and to withstand the required jacking and pumping pressures. It is usual to inject the grout while withdrawing the pipe from the maximum depth in well defined steps, ranging typically from 0.3 to 1m.

e. Cost

A split spaced grid pattern is utilized, with the grid pattern spacing and the volume to be injected dependent upon the required increase in density in the formation or the size of the void to be filled. Compaction grouting costs vary from as low as $5.00 per cubic meter of soil treated to over $50.00 per cubic meter (typically $20/m^3), plus mobilization and pipe installation costs. The variations in projects, drilling costs, mixtures, quantity injected, rate of injection, etc. makes this system particularly sensitive to price fluctuation. The cost of the grout alone is in the range of $60-100 per cubic meter.

f. Case Histories

Case 1 - Compaction Grouting Glenwood Canyon; Colorado D.O.T.[42]

Construction of Interstate 70 through Glenwood Canyon in Colorado included 2.4 km of retaining walls at the Bair Ranch interchange to support westbound and eastbound lanes. Exploratory borings prior to construction revealed a deep layer of compressible clay. Due to cost considerations, it was determined that the retaining walls could be constructed on spread footings if the walls were constructed in short segments to accommodate the 150 to 200 mm maximum settlement anticipated from consolidation of the clay. Wall panels were manufactured higher than was necessary in order to obtain final design grade.

With footing construction, panel erection, and backfilling complete, a program was implemented to monitor the anticipated settlements during the 8 to 12 months of primary consolidation and initiation of secondary compression. Although initial settlement was as anticipated, within a short time, rapid and erratic settlements occurred. Efforts to determine the cause included surcharging the wall to determine if settlements also occurred in areas where no surcharging was done, leading to speculation that the movement might be induced by wetting rather than loading.

Consolidation testing of undisturbed samples showed that the soils within 7 m of the surface were sensitive to wetting under load. A correlation was also established between rainfall/runoff and settlement. Compaction grouting was selected to densify the upper soils beneath the wall footings to decrease settlement potential and strengthen the soils against the effects of wetting.

Compaction grouting was performed through a single line of injection points at the toe of each wall in a staged, 'top down' process, each stage being allowed to set before the next stage was grouted. A total of 443 cubic meters of grout was injected through 233 points to completely treat the target soils. During the grouting operation, the grouting contractor monitored grout pressures and grout volume to limit the potential for uncontrolled heave.

Since the completion of the grouting program, and subsequent opening of the interchange to traffic, elevation surveys showed a maximum wall settlement of 6 mm as compared to the more than 350 mm of settlement that had occurred before grouting was implemented.

Case 2 - Compaction Grouting for Liquefaction; Imperial County, CA

The Worthington Road Bridge, which crosses the New River in Imperial County, CA, was an aging, wooden pile and frame structure that underwent severe deformation during the 1987 Superstition Hills (El Centro) Earthquake. This activity, centered nearby, had caused the fine sandy and silty deposits beneath the approaches to lose strength and liquefy. Sudden loss of soil strength combined with the approach fill surcharge and gradient towards the river caused the bridge abutment to slide, resulting in buckling of the deck.

Subsurface explorations revealed fill soils containing silty sand, clayey silt, and sandy clay deposited directly onto loose-to-very-loose fine sand and silty sand channel deposits. Ground water was present in all boring locations at depths of approximately 2.7 m.

The geotechnical engineer recommended compaction grouting as the most cost effective method of increasing in-situ density and mitigating future liquefaction potential. The treatment area extended for the width of the approach fills and to lengths of 91.4 m west of the west abutment and 76.2 m east of the east abutment. The grouting contractor drilled grout holes to firm stratum at a depth of 9.1 m on a regular, pre-determined grid pattern. Casing was installed to full depth of the holes and compaction grout was pumped in stages as the casing was incrementally withdrawn.

During the grouting operation, grout volumes, pressures and surface movement were carefully monitored and post-injection penetration tests ensured the quality of the grout columns. The geotechnical engineer calculated that the average overall soil density had been increased by approximately eight to nine percent, verifying the effectiveness of the compaction grouting program.

3.5 JET GROUTING

The jet grouting technique employs high pressure erosive jets of water or grout to break down the soil structure, removing varying proportions of soil and replacing them with a cement based grout. Soil particles not removed become mixed with the grout in-situ to form a treated mass. This grouting technology can treat soils ranging from clays to gravels.

a. Applications

Water control

Jet grouting has demonstrated its effectiveness in both horizontal and vertical water control under static water conditions as the grouted mass is less permeable than the in-situ soil. It can be used in contaminated soils.

Settlement Control

Jet grouting is used to provide foundation support through weaker, soft soils to more competent bearing strata.

Underpinning

Jet grouting has become a viable alternative to conventional underpinning since its introduction into the United States in 1987. Since jet grouting can serve two purposes, as both an underpinning element and as excavation support, it can have a considerable economic advantage. Jet grouting is a relatively safe operation; construction personnel are never required to work beneath the structure being underpinned, and there is no need to make load transfer connection between the existing foundation and underpinning units.

Excavation Support

Jet grout has the capability of being placed immediately next to and through the footings adjacent to the excavation, allowing for a vibrationless, safe, and designable method of excavation support. Jet grouting can also be used to place excavation cross bracing prior to excavation so that inward deflection of the excavation support is prevented.

Liquefaction Mitigation

Jet grouting transforms potentially liquefiable soils into a cemented mass.

b. Design of Soil Grouting Program

Jet grouting is particularly well suited to any area that has a high density of structures or utilities, where the ground is very variable, or otherwise not amenable to other grouting techniques, and where significant strength (say over 3 MPa) is required from the treated soil mass.

When jet grouting is used for underpinning and excavation support, one-meter-diameter columns are normally designed. Construction of the columns is sequenced such that no more than one meter of temporary bridging is required from the existing foundation. To evaluate the design feasibility of the underpinning and/or excavation support operation, the treated soilcrete strengths given in figure 35 may be used for the triple-fluid system as a guide. In general, it can be assumed that strengths produced by one fluid jet grouting may be at least as high, whereas two fluid strengths may be significantly lower, due to the air entrainment. The wall should be designed in accordance with standard design procedures taking into account the building loads that will be transferred through the foundation being underpinned and into the treated soilcrete underpinning wall. It must be emphasized that the final strength of the soilcrete will depend on the nature of the virgin soil (strength follows permeability), and the various operational parameters which are selected. For this reason, it is essential to conduct a preconstruction field test, to allow the actual column size, shape, homogeneity and strength to be demonstrated.

Program Design

Spacing: Jet grouted columns can be in the range of 0.8 m to 3 m in diameter, and interconnected overlapping columns are constructed in continuous rows in a primary/secondary sequence.

Soil Type	Soilcrete Unconfined Compressive Strength (kPa)
Clean Sands & Gravel	6900 - 17,000
Silty Sands	4800 - 10,000
Sandy Silts & Clayey Sands	3400 - 8300
Silts and Low Plasticity Clays	2100 - 6900

Figure 35. Range of typical soilcrete strengths. (Three Fluid System)

Grout Quantities: Jet grouting is less dependent on soil conditions than other types of grouting, and therefore the quantities reflect the design requirement (i.e. for underpinning design, the treated quantity and quality depends upon the load imposed and the bearing capacity permitted by the soil conditions.)

Grout Selection: For jet grouting, the grout typically consists of portland cement, and water with a w/c = 0.8 - 1.2. Bentonite and other materials, including additives may be used depending on the specific project, but should all be subject to the engineer's approval.

c. Construction Equipment

Typically, jet grouting equipment is either purpose built or specially adapted.

Drills

Continuous slow rotation and controlled, preset lift ability is essential to the jet grouting operation. The hydraulic rotary drills can be preprogrammed to control the rate of withdrawal, rotation, and therefore configuration of the jet-grouted element. All drilling and grouting parameters should be illustrated and recorded in real time via automatic parameter recording devices.

Pumps

Grout pumping units vary depending on the type of jet grouting operation. In single fluid jet grouting, high-pressure, high-volume grout pumps are required, capable of pumping 250-500 liters per minute, with pressures of over 50 MPa. The double fluid system uses the same pump with the addition of an air compressor. Triple fluid jet grouting uses a high pressure water pump combined with an air compressor, and a similar, or lower pressure grout pump, depending on the contractor's specific system.

Batching System

All three jet grouting systems require a high-capacity grout batching system composed of a silo, water supply, high speed high shear colloidal mixer, and an agitator tank. This system is usually semi automatic and computer controlled.

Jet Grouting Systems

The choice of grouting system and operational parameters is typically left to the contractor but subject to field verification. The choice is based on the type of in-situ soil to be jet grouted, cost and the required engineering performance parameters required. Table 4 provides an initial technical/economic assessment of grouting systems as a function of soil type. With respect to engineering characteristics, the single fluid systems will produce the smallest diameter (0.6 to 0.8 m) and strongest soilcrete in granular soils. The double fluid system produces larger diameter columns of lower strength while the triple jet system produces the largest diameter columns (2-3 m) of higher strength.

Table 4. Technical/economic assessment.

Soil Type	Technical Capability	Economic Preference
Gravels	Single, Double, Triple	Single
Clean Sands	Single, Double, Triple	Double
Silty Sands	Single, Double, Triple	Triple, Double
Silts	Triple	Triple
Clays	Triple	Triple

d. Cost

Jet grouting is designed to solve unique problems normally untreatable by other methods. The cost of a jet grouting can vary greatly depending on the complexity of the project and the depth of treatment. Recent costs on projects in clay at the complex Boston Central Artery project were about $200 per cubic meter of ground treated, and a typical range is $150-300 per cubic meter.

Figure 36 presents jet grouting prices for underpinning and excavation support based on evaluation of over 65 projects completed in the United States. The costs shown include mobilization, testing, and demobilization which ranged between $25,000 to $50,000. These items are project specific and will vary depending on project size, but typically would represent 5 to 15 percent of overall costs. These costs indicate a large variation and in general are for projects smaller than the Central Artery in Boston. If the extent of the program is well defined, jet grouting can be specified on a lump sum basis. Jet grouting may also be measured as: (1) mobilization, demobilization and testing as a lump sum, or (2) as a seepage barrier wall or underpinning project measured per square meter.

	Unlimited Headroom (<11m)	Restricted Headroom (2.4 - 3m)
Underpinning & Excavation Support 0.9 - 1.1m dia per/m of depth	$95 - $550	$490 - $650
Seepage Control 0.9 - 1.1m dia per/m of depth	$30 - $115	$30 - $200

* Soilcrete pricing includes mobilization, testing and demobilization

Figure 36. Range of jet grouting prices.

e. Case Histories

Case 1 - Scour Repair of Salt River Canyon Bridge; Arizona D.O.T.

The Salt River Canyon Bridge is located on US Route 60, 193 km east of Phoenix, AZ. Heavy rains pushed the river's water more than 9.1 m above normal levels, high enough to surround the new bridge abutment. Most of the canyon walls underneath the bridge are sheer vertical rock faces. An area immediately adjacent to the new abutment, thought to be a solid portion of the canyon wall, scoured out, leaving a 6.1 m-wide gouge in the rock as shown in figure 37. The gouge threatened the stability of the new bridge and washed away the only access road along the river's edge. The gap became visible when the river waters receded.

The grouting contractor responded to an emergency call from Arizona DOT to repair the scour problem. The task required placing a retaining wall between the rock faces on both sides, thereby protecting the new abutment. Triple-fluid jet grouting was selected as having the capability to solve the problem within the response time needed. Interconnecting jet grouted columns were constructed below the normal river level to fill the gap and act as an arched retaining wall to stabilize the scour zone as shown on figure 38. Following this work, a concrete retaining wall was founded on top of the grouted mass to fill the gap above the water level. Restoration of the access road was then completed.

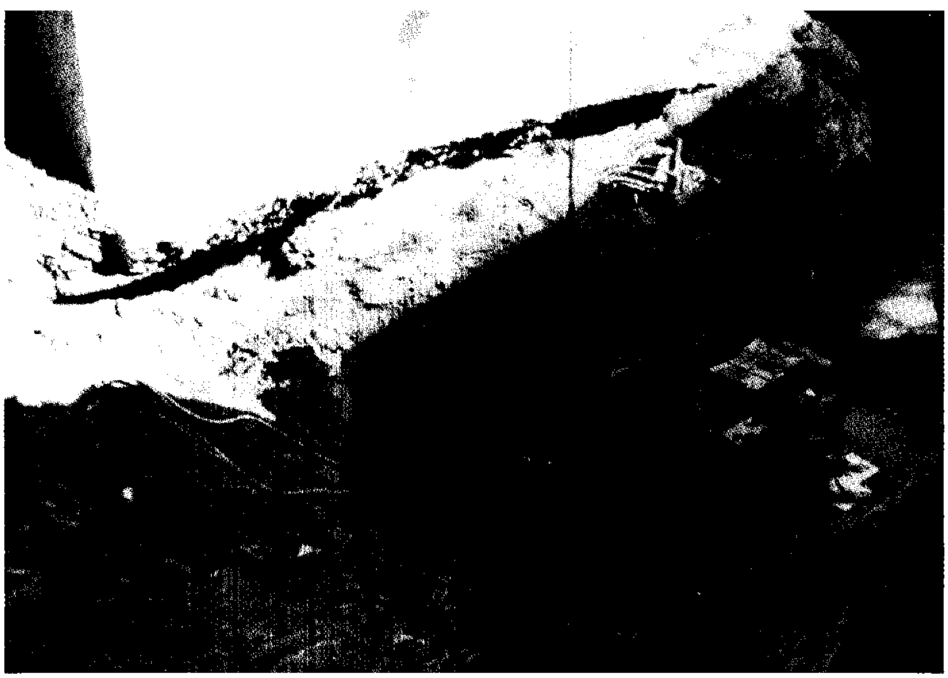


Figure 37. Salt River bridge scour.

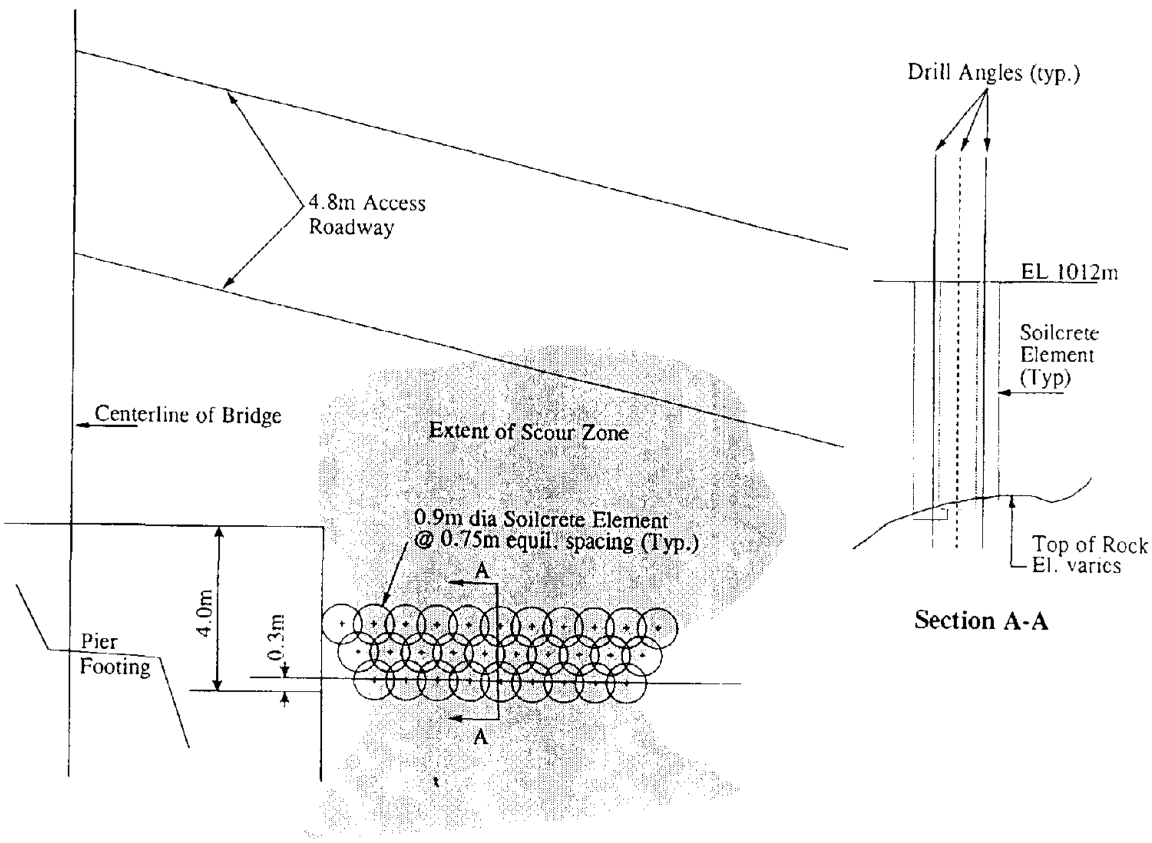


Figure 38. Scour zone after stabilization.

Case 2 - Excavation Support of East End Relief Sewer; Honolulu, HI

The alignment of a new sewer utility proposed by the City of Honolulu's Department of Waste Water Management placed the utility beneath a number of Honolulu's crowded streets and busy intersections. Though conventional open-cut excavation with braced sheet pile support was employed by the general contractor for most of the work, the intersection of Auahi Street and Ward Avenue contained numerous existing utilities that could neither be removed nor relocated, and that prevented the general contractor from performing his sheeting operations.

The general contractor and owner agreed that an alternate excavation support system was needed in order to proceed with work in the intersection. The grouting subcontractor proposed using its jet grout system to install a continuous wall around and beneath the existing utilities in order to provide not only excavation support but also groundwater control and underpinning as shown on figure 39. By accurately locating the utilities and pinpoint drilling, it would encapsulate the utilities in a grouted wall. The new utility's invert at the intersection was 5.75 m below existing grade.

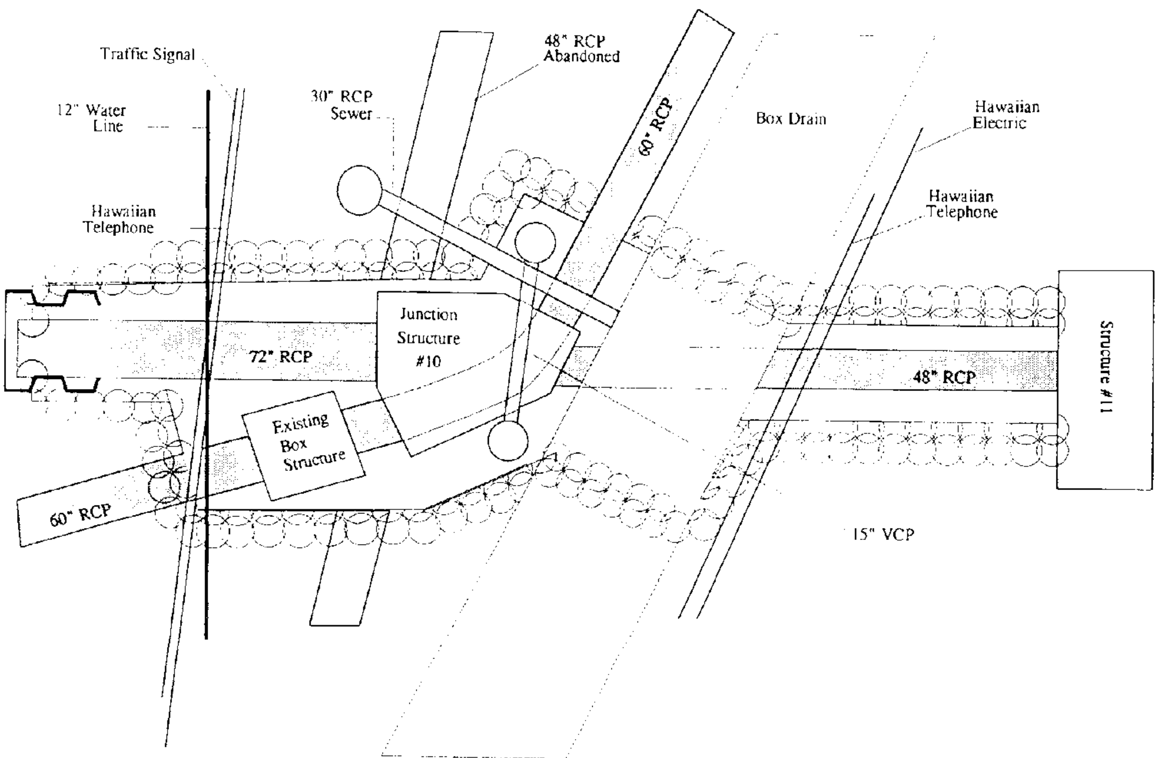


Figure 39. Soilcrete wall around utilities

The soil profile consisted of very loose silty sands to a variable depth of approximately $\pm$ 6.3 meters, where hard coral was encountered. The groundwater table was only 0.6 m below the existing grade. However, dewatering of the intersection was a difficult proposition due, in part, to gasoline contamination in the soils.

Among the utilities present under the intersection was a 6.2 m wide reinforced concrete double box drain, 0.75 m and 1.5 m diameter reinforced concrete sewer pipes, water lines, telephone lines, and electrical lines. Each utility was visually located prior to jet grouting and the soilcrete columns were angled as necessary, to ensure adequate closure of the soilcrete walls as shown on figure 40. The reinforced concrete double box drain was first cored, and casing was set and sealed in place to prevent the loss of spoils into the drain. The casing was removed after the columns were grouted and, following completion of the work, the general contractor installed a reinforced collar around the box drain along the alignment of the core holes to re-establish structural integrity of the box drain.

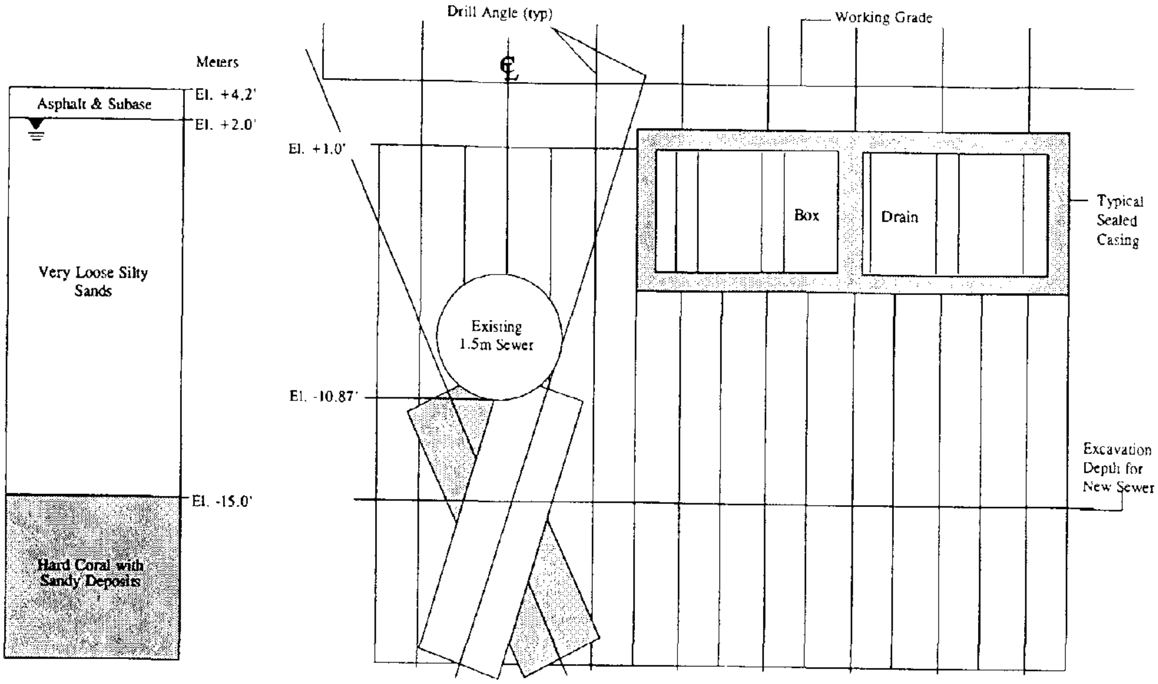


Figure 40. Soilcrete columns angled to protect existing utility.

To minimize disruption to traffic patterns, all of the work was performed at night. After installing the soilcrete wall and during excavation, one level of bracing was required in the trench. This bracing was temporarily removed for pipe to be placed and completely removed after backfilling of the trench had begun. The excavation proceeded without difficulty, without disruption to the existing utilities, and without the need for continuous dewatering.

3.6 SOIL FRACTURE GROUTING

Soil fracture grouting is the most recent grouting technology having been introduced in the United States in the early 1990s. Its primary use is to raise settled or settling structures to their original elevation in a highly controlled manner and increase the carrying capacity of soft and/or loose soils.

a. Applications

Raising Settled Structures

Soil fracture grouting has the ability to raise sensitive structures with a high degree of control, coupled with state-of-the-art instrumentation monitoring.

Settlement Control

Soil fracture grouting controls vertical movement by predesigned fracture injections of particulate slurries. It can be used to relevel structures founded on soft, cohesive soils or to maintain structures during tunneling, in which case it is referred to as "compensation grouting."

Underpinning

Soil fracture grouting is a proven system of underpinning throughout Europe and has recently been introduced in North America.

Soil Reinforcement

Fracture, or "lense" grouting has been used in California to reinforce clayey soils subject to lateral movement. Fibers are added to the grout to provide tensile strength.

b. Design of Grouting Program

Soil fracture grouting works best in soils that are not free draining, but it can be applied to all soil types.

Because the process requires that the soil is fractured and not permeated, soil fracture grouting may be used in most soil types ranging from weak rocks to clays. Cementitious grouts are injected in a very controlled fashion so as to create a reinforcing matrix of grout. In this way, both the strength and the stiffness of the soil are increased, and tunnel induced movements can be compensated for.

Program Design

Spacing: U.S. experience with soil fracture grouting is relatively limited. Previous project experience indicates potential spacing between pipes on the order of 1-2m, although most examples involve "fans" of grout holes which are therefore not parallel within their plane.

Grout Quantities: Grout is injected via sleeve port pipes, permitting multi-injections from each port. The actual quantity injected will depend on the soil type being grouted, the purpose of the program, and the type and nature of the tunnelling method, when used in compensation grouting. However, individual sleeve volumes of 30-135 liters can be expected.

Grout Selection: Portland cement is the base for multi component grouts of special rheological and settling properties. Typical other materials may include bentonite, flyash and chemical additives such as accelerators and viscosifyers.

c. Construction Equipment

Pumps

Positive displacement grout pumps should be capable of controlled pumping pressures up to 6,500 kPa and pumping rates between 4 and 40 liters per minute. Each grout pump must be capable of displaying pressure, and injection rate and volume. In line pumps for the injection of additives must be capable of maintaining component ratios constant over the specified grout-pumping ranges.

Injection Pipes

Injection pipes are typically steel sleeve port type with injection points at regular intervals as required for adequate grouting capability for the project. Flexible sleeves over injection ports must be of material compatible with grout and chemicals. These pipes are typically installed in horizontal planes for compensation grouting, and are vertical for lense grouting for soil reinforcement.

Packers and Hoses

Packers should be pneumatically or hydraulically activated double packers ("straddle packers") capable of isolating individual injection ports. Packers must be able to create a tight seal at an injection port that must withstand the full range of pumping pressures. Hoses for grout must be sized to withstand appropriate pumping pressure and flow and must also be of sufficient rigidity to move packers along the full length of grout pipes.

d. Cost

There is too little project experience in the United States to gauge the budget cost of soil fracture grouting. It is recommended that grouting specialists be contacted to provide feasibility and cost data for any potential project.

Alleviating Tunneling Subsidence; CNS Sarnia Tunnel[34]

For the construction of the St. Clair River tunnel, part of the Canadian National Railway system, compensation grouting was used for the first time in North America to protect numerous sensitive above-ground structures and buried utilities during soft ground tunneling below.

The St. Clair River tunnel is a 9.5 m diameter tunnel that was built to replace the existing tunnel which was structurally sound but of a capacity inadequate to accommodate double-stack container cars (figure 41). The tunnel was to be driven under shallow (4.5 m minimum) cover through soft clays, and maximum predicted settlements caused by the tunnel boring machine were as much as 130 mm. Since the tunnel was to pass under a number of buildings owned by the Esso Imperial Oil Company (figure 42), including a research building housing sensitive equipment, reinforcement of the soft clays and settlement compensation was required during the tunneling operation.

To protect the three-story, reinforced concrete research building, horizontal arrays of sleeve port grout pipes were drilled and installed under the building from two 9 m deep shafts, as shown in figure 43. The 4.5 m diameter shafts as shown in figure 44 were installed on either side of the building, diagonally opposite one another. The grout pipe spacing and position between the crown of the tunnel and the existing building foundations were designed by the grouting contractor based on the predicted settlement trough of the tunnel as well as the existing soil conditions.

By repeatedly injecting small volumes of controlled rheology grout through individual ports along the grout pipes, a supporting skeleton of grout lenses was created in the clay below the building before the tunneling machine passed underneath. This phase may be called "preconditioning." Grout injections were carried out using a computer controlled grout-pumping unit capable of six simultaneous injections while recording pressure, flow, and volume. Coupled with the grout pumping unit, a real-time remote movement monitoring system was employed to measure building movements on the order of 0.4 mm.

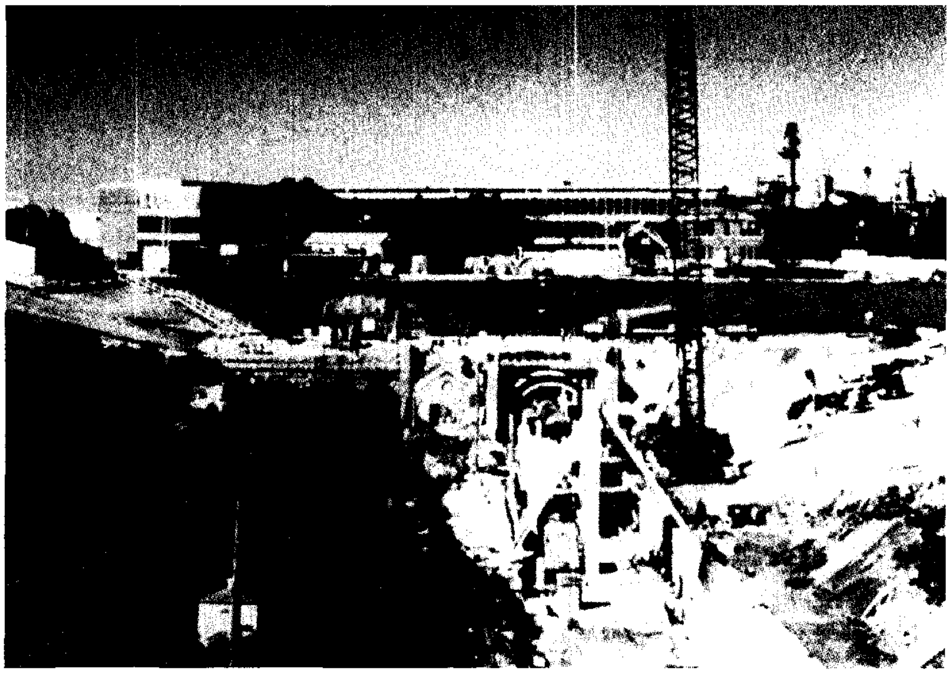


Figure 41. Existing tunnel, Sarnia, Ontario.

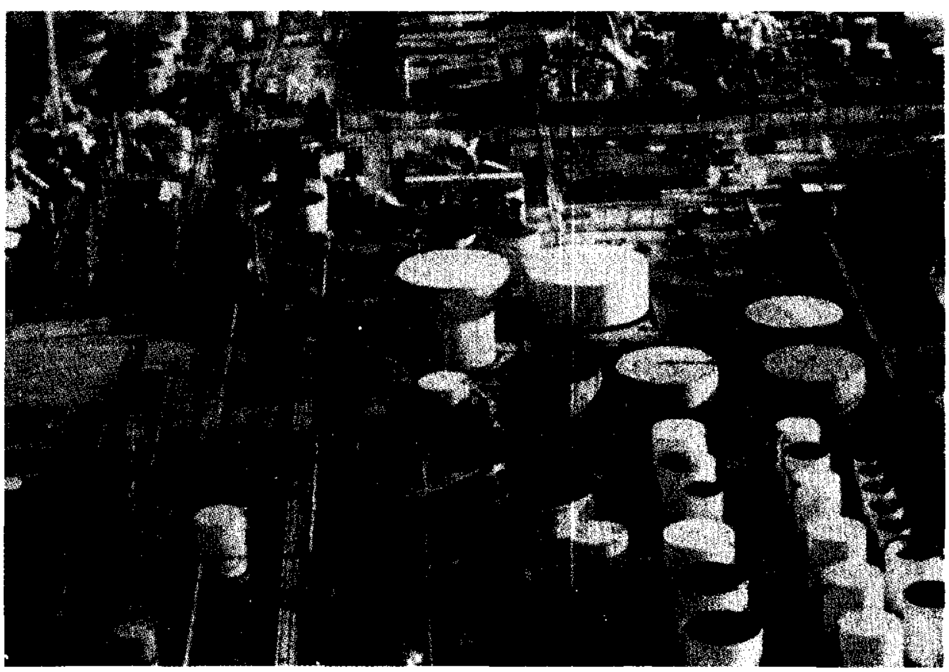


Figure 42. Aerial view of proposed tunnel site.

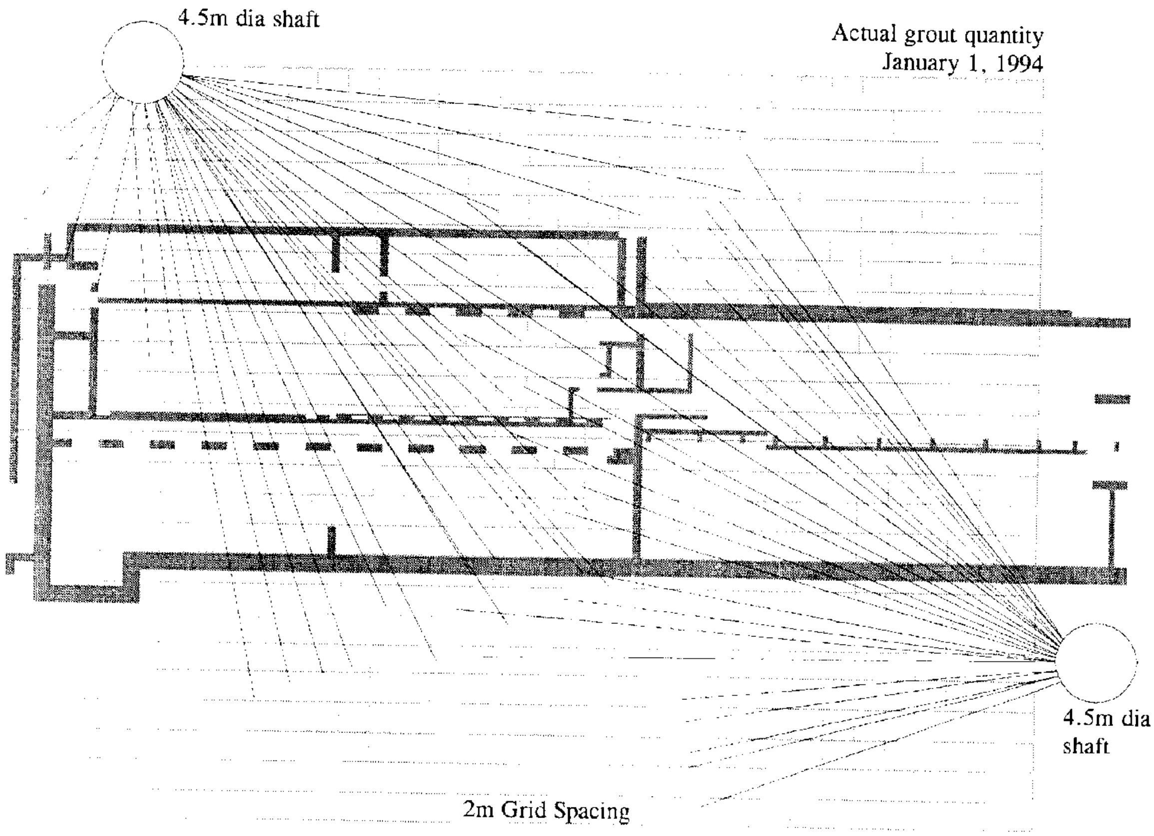


Figure 43. Grout pipe locations for tunnel site.

As the 9.4 m diameter tunnel boring machine passed within 9 m of the building footings, further precise grout injections were made in response to slight building settlement in order to heave, and thus maintain the position of, the building. This is the actual "compensation" phase.

Overall building movements were controlled during the work well within the specified criteria of less than 1/1500 angular distortion, and 6.3 mm of overall settlement.

In addition to the settlement protection of the research building, the soil fracture technique was also used successfully under several other structures including a series of 0.75 m diameter cast iron service water pipes that were critical to the refinery's operation , an electrical substation, and a river water pump house.

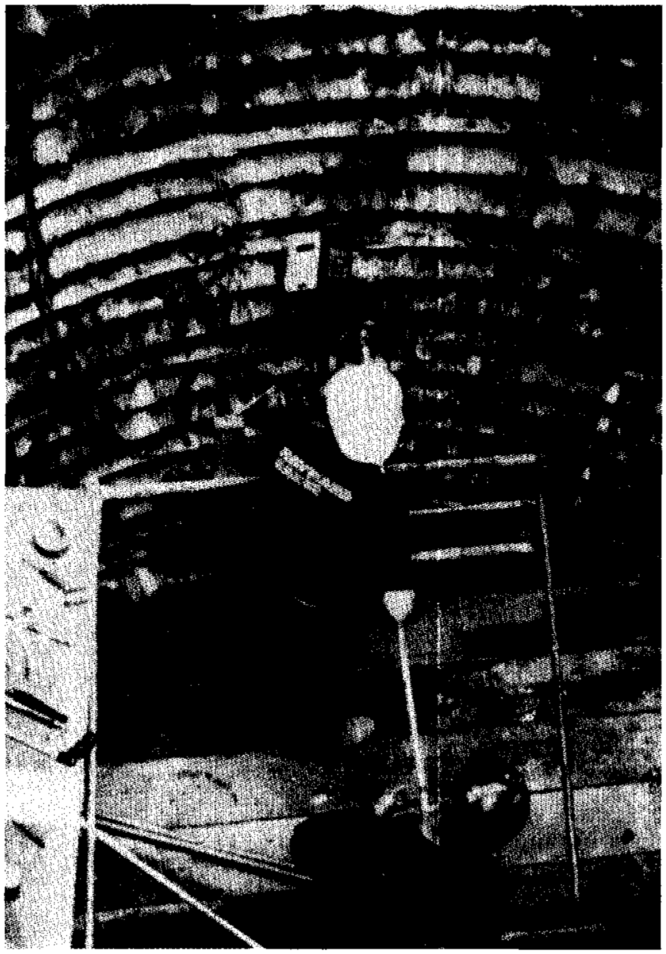


Figure 44. Access shafts used to inject grout.

3.7 LIME INJECTION GROUTING

Lime injection grouting is used to control expansive clays beneath structures.

a. Applications

Expansive soil has the potential for shrinking and swelling under changing moisture conditions. The resulting deformation is usually of an uneven pattern and of such magnitude as to cause extensive damage to overlying structures and pavements. The deformation typically occurs in the form of heave. It is estimated that in 1970, residential losses from expansive soils were in excess of $800 million. By the year 2000, total losses are estimated to reach almost three billion dollars.[43]

At least half the total dollar losses contributed to expansive soils are related due to pavement damage.[44] Pavements are particularly susceptible because they are lightweight and extend over large areas. In highway construction, it is typically not economical to bypass or replace expansive material. Subgrade soil treatments are therefore commonly employed to minimize soil movement. These treatments include construction of moisture barriers, control of placement density and moisture content of base course and compacted subgrade material, mixing the in-situ soils with lime or other stabilizing agents, and lime injection grouting.

Typical applications include the rehabilitation of cracking masonry walls, slope stability failures, pavement and slab failure, landfill stabilization, water control, and railroad embankment stabilization.

b. Design of Grouting Program

Where cyclical wet-dry conditions are present, expansive soils can cause distress to the structures in the area and the ground on which they are founded. Lime slurry pressure injection (LSPI) has been proven to be a viable solution to these problems.[45] Lime injection can also be utilized for new construction where instability problems exist or are anticipated.

A slurry pressure injection program consists of the injection of hydrated lime slurry, usually mixed with fly ash as an inexpensive filler, into foundation soils that have failed or are failing. The lime slurry fills in cracks, voids, root holes, etc. and reacts pozzolanically with the soil to create a matrix of reinforced earth.

Grout Quantities

A typical water-lime ratio is about 0.3 to 0.4 by weight. Lime injection grout quantities, as in compaction grouting, will depend on factors such as soil type, void space, etc., but the lime-water ratio should never be above 0.5. When working on railroad subgrades, approximately 90 metric tons of lime slurry per kilometer of railroad are typically used (about 295,000 liters of slurry).

Grout Selection

Usually, the lime used is either high-calcium hydrated lime such as calcium hydroxide or dolomitic (calcium hydroxide combined with magnesium oxide) hydrated lime. However, it is important to note that laboratory tests should be conducted to pinpoint the effectiveness of each material.[45]

Other materials are typically:

- Water - Should be pure, clean, and free of pollutants such as high acid or sulfate content, which could interact with or decrease the effectiveness of the lime. The sulfate/sulfite content should be less than 500 mg/l and the pH 6.5 or greater. Brackish water and water high in chloride ions have been employed for slurry preparation, but caution should be taken as these additives may cause corrosion of equipment and flash setting of the slurry.

- Surfactant - To reduce water tension and to promote slurry penetration, a non-ionic wetting agent is usually added at a rate of one part surfactant to 3500 parts water.

- Fly Ash - An artificial pozzolan, high quality fly ash containing less than 2 percent organic carbon is commonly used in lime injection applications. Comprised of mostly silica, fly ash reacts with lime to form siliceous cementing compounds. If alumina is present, calcium aluminate forms. If the organic carbon content is higher than 2 percent, higher percentages of lime and accelerators must be added to create a beneficial reaction. Laboratory tests should be performed on all fly ash to pinpoint the exact content.

c. Construction Equipment

Injection Vehicle

The injection vehicle should be capable of forcing injection pipes into the soil with minimum lateral pipe movement to prevent excessive blowbacks and loss of slurry around the injection pipes. The vehicle should be a rubber tired or tracked machine suitable for the purpose.

Pumps and Tanks

Slurry pumps should be capable of pumping at least 11 cubic meters per hour at 350-1,400 kPa.

Slurry tanks should have a suitable mechanical agitation system to ensure proper mixing and uniformity of slurry.

d. Cost

Lime slurry pressure injection is usually about one-half the cost of a shallow and backfill solution. For example, in the Dallas/Fort Worth area in 1985, removing one meter of material from a 4,180 m^2 site and replacing it with one meter of select material in compacted 150 mm to 200 mm lifts cost about $50,000. The same results would be obtained by a double injection of lime slurry to a two meter depth for approximately $20,700.[45] These costs are equivalent to about $12.50 per square meter for over-excavation and backfill and $5.00 per square meter for lime slurry pressure injection.

e. Case Histories

Case 1 - Highway Stabilization of Harlington Loop 499; Texas D.O.T.

Planned upgrading of Loop 499 in Harlington, Texas from two to four lanes required construction across a 8.5 m-deep landfill. The landfill had previously been paved soon after closure and had subsequently settled over 1.25 m. Concerned that a potential for significant settlement still existed that would threaten the integrity of the new construction, Texas DOT solicited bids for densification/stabilization of the landfill, with the requirement that the existing two-lane highway stay open to traffic during the work. Dynamic compaction, a proven landfill densification technique, was not a viable option since this would require road closing. Removal and replacement or other densification techniques, such as compaction grouting, were economically unacceptable. Lime fly ash injection, proposed by the specialty contractor, was accepted as a stabilization method that would fill the voids and strengthen the landfill mass through a network of reinforcing seams of cemented lime fly ash slurry. The technique had proven useful in prior landfill applications. It had also been used extensively, particularly in Texas, to stabilize and reinforce soils that contained similar voids as would be encountered in the landfill.

Grout injection points were drilled through the settled pavement to the full depth of the landfill on a 0.76 m grid pattern and lime fly ash slurry injected at a target rate of 2.8 liters per cubic meter of ground area. Due to the heterogeneous nature of the landfill, injected quantities were not uniform across the site, with some areas taking no slurry and others taking well beyond the target amount, depending on the density of the landfill material at a given location. The fluid slurry travelled through the material, filling any encountered voids and spaces between the debris, forming a solidified structural network and thus minimizing future consolidation. However, one year after completion of the highway upgrade, signs of slight settlement were observed. This was uniform and no pavement cracking occurred. The settlement was theorized to be attributable to a number of reasons: a water line that crossed the site and had previously been leaking might have begun to leak again, the weight of the large volume of injected lime fly ash slurry might have caused settlement, or the problem might be at a greater depth than the stabilized landfill. Ongoing evaluation of the distribution and time rate of settlement have concluded that settlement has stopped.

Case 2 - Slope Strengthening of Missouri Flood Walls[43]

Part of the Corps of Engineer's Missouri River Levee system near Brunswick, MO required rehabilitation due to several failure slides that had damaged approximately 400 linear meters of slope, as shown in figure 45, significantly reducing flood protection. Geotechnical studies revealed that 4.6 linear meters of the embankment, including the damaged portion, was comprised of highly plastic, montmorillonite clay placed in an uncompacted, saturated condition on saturated soils.

A specialty contractor was retained by the Corps to construct a test section using lime/fly ash injection to determine if this technology would be effective in stabilizing the embankment. Independent laboratory tests on treated and untreated samples showed that the lime/fly ash injection had reduced the soil's plasticity and increased its strength by 400 to 1500 percent in 35 days. Several months after the test section was completed, the levee had experienced failure in other areas, including the untreated control section, but the treated section remained intact.

Based on these results, the specialty contractor was retained to stabilize the levee. Crawler tractors equipped with four injection probes each (figure 46) treated both sides of the levee to a depth of 3.0 m through primary and secondary injection points. Test pits were dug every 150 m to ensure the effectiveness of the injection program in penetrating into voids, failure planes, and tension and desiccation cracks. In the summer of 1993, severe flooding caused extensive damage, washing out most levees in the area. However, the lime/fly ash treated levee remained intact, with only minimal damage.

Case 3 - Lime/Fly Ash Injection to Rectify Subgrade Problem; Buckeye, AZ[44]

Reconstruction of a 1.6 km section of rural Beloat Road in Maricopa County, AZ was stopped when the placement of an aggregate base course caused the in-place subbase to break up. County officials discovered that this section of road was underlain by a very water-sensitive silt. During rainy periods, water from irrigation ditches and flooded fields along one side of the road and from a deep drainage ditch along the other side had saturated the silt. Consequently, it could not support the additional material loads.

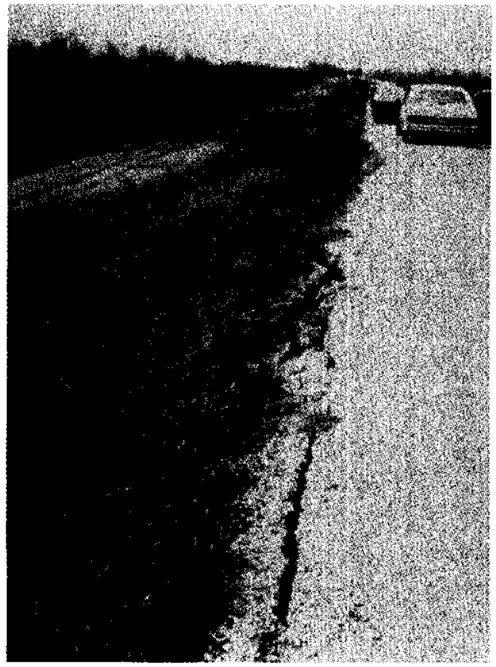


Figure 45. Damaged slope due to failure slides.

Figure 46. Modified crawler tractors used to inject lime slurry.

Attempts to remedy the problem included, in turn, excavating the subbase and replacing it with river rock, and placement of geogrid mats over geotextile filter fabrics in the subgrade and subbase. Both of these approaches were unsuccessful. A third alternative, proposed by a grouting contractor in cooperation with the consulting engineer retained by the county, was to stabilize the soils by lime/fly ash injection. This option was accepted as being technically achievable, cost effective, and able to be accomplished in the short time frame necessary to allow the roadwork to be completed.

Although lime/fly ash injection for subbase stabilization had been previously limited to depths of 0.3 m or less, the grouting contractor and the consulting engineer determined that a slurry composed of four parts fly ash to one part lime would achieve the desired improvement to the required depth of 1.2 m. The slurry was injected into the subbase at 1.5 m centers across the width of the roadbed, followed by secondary injections to split the primary spacing. The treatment created a moisture barrier in the subbase, and the network of lime/fly ash seams served to increase the shear strength of the soil as well as to reduce the moisture content by forcing out the free water. After the injection was completed, the subgrade surface was disked to work the excess slurry into the ground and create a good working subgrade.

This operation was completed in 17 days and produced excellent results. A continuous bearing surface and moisture barrier was created atop the subgrade which, after curing, achieved strengths of 2,000 to 2,750 kPa, easily supporting the construction and traffic loads.

CHAPTER 4

ROCK FISSURE GROUTING

For transportation facilities, rock grouting techniques typically have limited application, although there are circumstances in transportation remediation or construction where rock grouting might be considered. This chapter discusses applications, advantages and disadvantages, design considerations, program design and construction, and costs. A case history is presented illustrating rock grouting for tunnel water control.

4.1 APPLICATIONS

By far the most common use of rock grouting today is in dam and tunnel construction and rehabilitation, especially for structural stability and groundwater control. However, the technique is by no means limited to these structures and can be applied on any project where there is a hydraulic or structural requirement to fill the fissures in a rock mass. For transportation facilities, potential applications might include shaft repair and the remediation of deteriorating, road or railway tunnels, and the stabilization of rock slopes.

4.2 DESIGN CONSIDERATIONS

Before deciding if grouting is appropriate for a particular site, a thorough subsurface investigation should be conducted. Several types of rock formations may exist, including weak or loose rock, rock with stress fractures, rock with large voids and rock with open fractures and/or possessing high permeability.

Often a design phase test program is warranted to determine the effectiveness of a rock grouting program. Based on the data obtained from this program, a grouting design, and program cost estimate can then be developed.

a. Advantages/Disadvantages/Limitations

The alternatives to the repair of weak or permeable rock range from alternative technologies, to costly removal and replacement, to abandoning the site. In most instances, none of these options is practical and rock grouting is a commonly used, engineered solution.

However, over the years, experience has shown that it can be difficult to pre-assess the cost of a rock grouting program. Site geology can be extremely complex, with widely differing subsurface conditions existing within the site boundaries. Even when a test program is performed, the statistical results may still not be sufficient to determine project costs with a reliable degree of accuracy.

In addition, poor field practices may lead to:

- Inducing uplift and damage to foundations resulting from excessive pressures.

- Improper plugging of foundation voids by thickening the mix prematurely or by unsuitable injection methods.

- Improper hole spacing or orientation of grout holes.

These can be rectified by utilizing knowledgeable and experienced geotechnical and inspection personnel to design, supervise and inspect the drilling and grouting operations.

b. Feasibility Evaluations

The main consideration for rock grouting in sealing cracks and fissures in rock or injecting grout for either water control or structural improvement purposes is the grain size of the particulate grout compared to the width of the rock fracture to be grouted. Mitchell presented groutability ratios for rocks as follows:

For Rock: N_R = fissure width / (D_{95}) grout
$N_R > 5$: grouting consistently possible
$N_R < 2$: grouting not possible

where "D" is defined as the grout diameter, and the subscript is the percent finer.[54]

The rock characteristics cannot be changed, but the fineness of the particular grout can be controlled and its rheological properties carefully engineered so that the N_R number for rock grouting feasibility is now nearer 3 than 5. It is vital to measure the in situ permeability of the rock mass in advance since this fundamentally influences the design, construction and verification processes.

c. Environmental Considerations

Care must be exercised when performing grouting in rock where the grout could leak into a body of water. The depletion of oxygen by the grout or the effect on the pH of the water could lead to a fish kill.

4.3 DESIGN OF A ROCK GROUTING PROGRAM

a. Problem Definition

Grouting may be used in new construction or as remedial treatment for repair of earth and rockfill dams, concrete dams, tunnels, and shafts. When the grouting program is properly designed and implemented, effective results can be achieved. It must be realized that regardless of how well conceived and designed the grouting program is, its success depends upon the field techniques used and upon good judgment by the field personnel.

Subsurface Investigations

Subsurface investigations are designed to assess the need for grouting and to provide information for design and construction monitoring of the grouting program. Investigations for grouting may include any geological or geotechnical method normally used for regional and site specific investigations and should be of sufficient detail to eliminate major surprises. Major components of the subsurface investigation include leakage potential, areal and structural geology, in-situ stress conditions, hydrogeology, geochemistry, and compatibility of in-situ and grouting materials. Rock mass discontinuities, especially frequency and aperture, are vital to record, as is the in situ permeability of the rock mass. The presence and characteristics of anomalous conditions are ascertained and appropriate treatment planned. Grout takes, mixes, procedures, and pressures are best determined or estimated by conducting a grout test program at the site to provide statistical information on average residual permeabilities which can be achieved by grouting.

Numerous case histories have demonstrated the necessity for thorough geological exploration prior to grouting and for continuous assessment during grouting. Subsurface investigations for design of grouting treatment have more often than not been limited by economic considerations or a failure to recognize their importance.

Purpose and Types of Treatment

Rock grouting with particulate materials normally falls into one of the following categories;

Curtain grouting is the drilling and grouting of one or more lines of grout holes to produce a barrier to seepage. The curtain usually extends into materials judged acceptably impermeable.

Area grouting (also known as "blanket" or "consolidation" grouting) normally consists of grouting a shallow zone in a particular area, utilizing grout holes arranged in a pattern or grid. Its purpose is to mechanically improve fractured and jointed rock. Deeper area grouting is sometimes done to grout specific geologic conditions such as fault zones or to consolidate subsurface materials at shaft, or buried structure, locations.

Tunnel grouting may be used to fill voids behind tunnel liners (contact grouting), treatment of material surrounding the bore, or seepage control. Pre-excavation grouting may be required for ground strengthening and water control on some projects.

Backfilling of subsurface exploration boreholes and grout holes is important to maximize structural stability or for water control or to prevent passage of contaminants to underlying strata.

b. Program Design

The precise goal of the program must be clearly stated. This may be a required decrease in permeability, as measured by post-grouting tests, or an increase in rock mass strength or homogeneity, as illustrated by core-sample testing or load testing.

The design of any grouting program normally consists of defining the areal extent of grouting, the number of rows of grout holes required, the grout materials, initial hole spacing and diameter, quantities of grout, grouting equipment methods and parameters, and developing performance requirements.

Most rock fissure grouting is done with particulate grouts. Cements in common use are described in chapter 2. The exact mix formulation must reflect the fluid and set properties that are required to enhance penetrability, and to provide a durable product.[45A]

c. Performance Monitoring

Detailed performance monitoring and evaluation is an integral part of the grouting program. Evaluation of daily records of drilling, pressure testing and grouting operations enables any necessary technical changes to be made as the project progresses. For example, the geologic profile that is developed from test boring data, and upon which the design of the rock grouting program is based, may not accurately reflect the subsurface conditions overall, since the number of exploratory test borings made on a project is too often limited by cost considerations. During the drilling process, deviations from the anticipated rate of progress and rock or mud cuttings recovered are indicators of an unexpected subsurface condition. This information serves to "fill in the gaps" between test borings, allowing a more detailed geologic profile to be developed. All of this information is included in the as-built report.

Computerized monitoring, recording and analysis of grouting operations provides instantaneous, accurate information on progress at any given location. This allows immediate input to the field construction crews as to progress and necessary changes. First used in the United States in 1983 by the Bureau of Reclamation at Ridgeway Dam in Colorado, computerized grout monitoring was highly successful, and is now standard practice as a monitoring and control mechanism in North America.[46,47]

The performance of the grouted rock mass must also be monitored with time. For example, if the goal is water tightness, seepage flows and pressures should be monitored. For blanket grouting, structural movements should be monitored, and so on.

4.4 CONSTRUCTION METHODS

a. Drilling and Flushing Methods

These are usually the choice of the contractor, and are discussed in chapter 2. Upon completion of drilling, the grout hole is washed or flushed until all drill cuttings and turbidity are removed. This is a separate operation from pressure washing, which is performed with the pressure testing rather than the drilling equipment.

Pressure washing and pressure testing are conducted immediately before pressure grouting operations are begun for the hole. Pressures used for pressure washing and testing should not exceed the maximum allowable grouting pressures. Washing continues as long as clay or washable materials are being removed from an interconnected hole or surface leak or as long as the rate of water injection increases at a given pressure. A clay dispersant can be used. A pressure test with clean water is performed after pressure washing, either at a constant pressure, or at multiple pressures.[3]

b. Grouting Methods

Rock grouting practice largely follows traditional lines. There are three basic methods used for grouting stable rock masses:

- Downstage (Descending stage) with top hole packer;
- Downstage with down hole packer as shown on figure 47; and
- Upstage (Ascending stage) as shown on figure 48.

The advantages and disadvantages of upstage and downstage methods are summarized in table 5. The competent rock available on most dam sites is well suited for upstage grouting and this has historically been the most common method. Downstage methods have recently had more demand reflecting the challenges and difficulties posed by more difficult site and geological conditions at remedial and hazardous waste sites.

In some cases of extremely weathered and/or collapsing ground conditions, even descending stage methods can prove impractical, and the MPSP (Multiple Packer Sleeve Pipe) Method is now the method of choice as illustrated in figure 49.

Regarding grouting pressures, there are various "rules of thumb", summarized in figures 50 and 51, and ranging from 1 to 4 times the theoretical weight of rock above the injection point. Many factors will dictate the site specific choice, such as those geological and structural, but the maximum safe pressure can be confirmed in preconstruction testing.

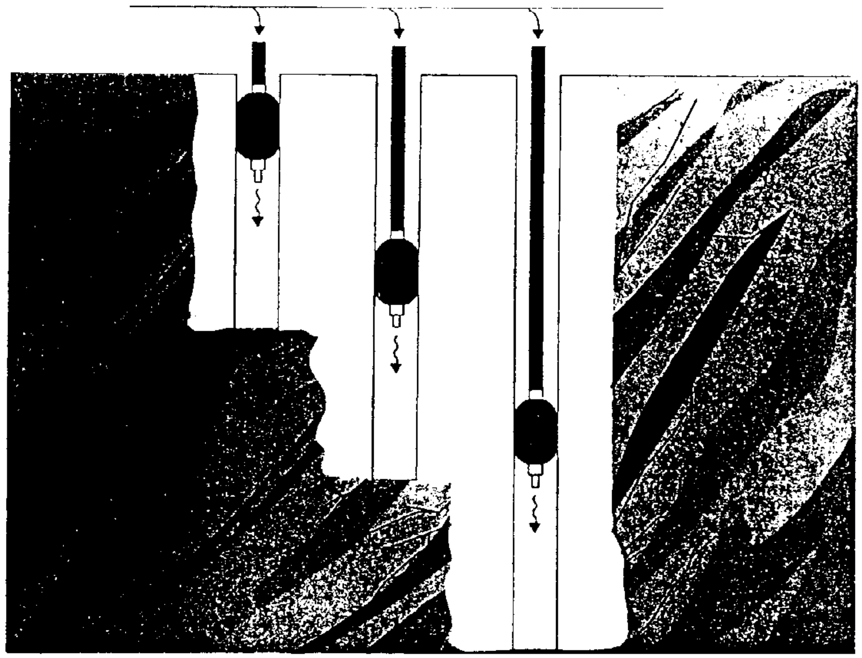


Figure 47. Downstage grouting.

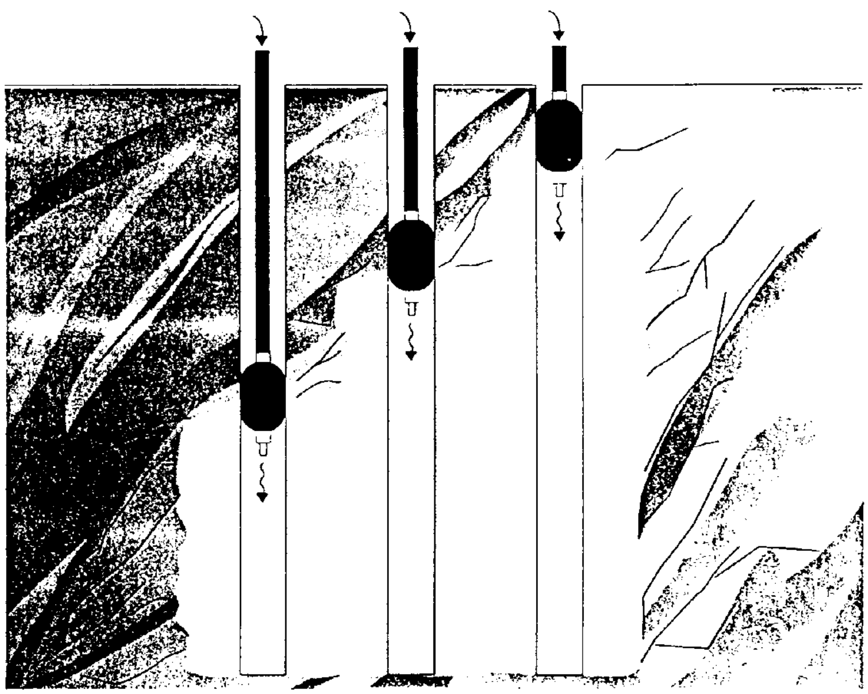


Figure 48. Upstage grouting.

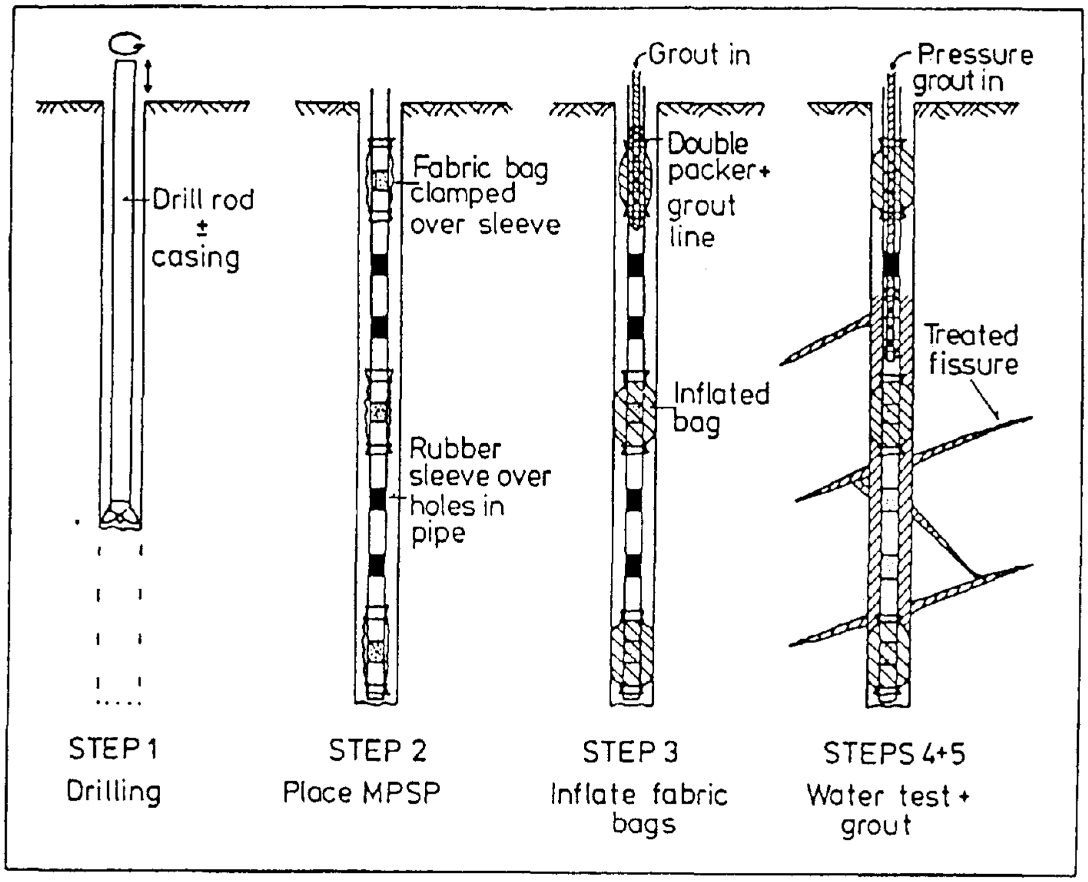


Figure 49. Multiple packer sleeve pipe process.

c. Grouting Equipment

As described in chapter 2, the basic components for a grout plant are a mixer, agitator/storage tank, pump, monitoring and control instrumentation and a wide range of ancillaries such as valves, pipes, fittings etc. The exact choice is site specific and within the range of available equipment which can vary from small mobile machines to full scale grouting stations. References 3 and 7 provide complete listings, descriptions and operating characteristics of presently available equipment.

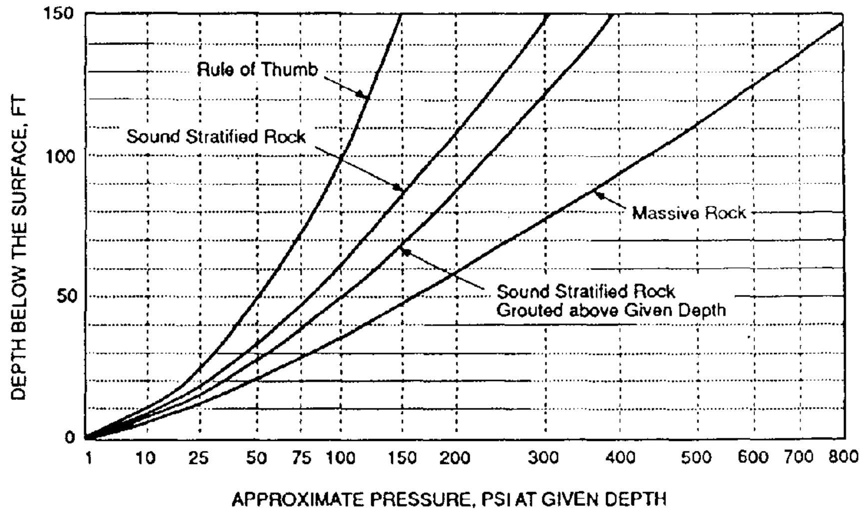
4.5 COST DATA AND BID ITEMS

a. Bidding Methods

Rock grouting may be performed as part of a general construction contract or under a separate contract. Typically, pre-construction grouting would be performed under the general contract while post-construction remedial grouting would be under a separate contract.

Table 5. Major advantages and disadvantage of downstage and upstage grouting of rock masses.

	DOWNSTAGE	UPSTAGE
ADVANTAGES	1. Ground is consolidated from top down, aiding hole stability, packer seating and allowing successively higher pressures to be used with depth without fear of surface leakage. 2. Depth of hole need not be pre-determined: grout take analyses may dictate changes from foreseen, and shortening or lengthening of hole can be easily accommodated. 3. Stage length can be adapted to conditions as encountered to allow "special" treatment.	1. Drilling in one pass. 2. Grouting in one repetitive operation without significant delays. 3. Less wasteful of materials. 4. Permits materials to be varied readily. 5. Easier to control and program. 6. Stage length can be varied to treat "special" zones. 7. Often cheaper since net drilling output rate is higher.
DISADVANTAGES	1. Requires repeated moving of drilling rig and redrilling of set grout: therefore process is discontinuous and may be more time consuming. 2. Relatively wasteful of materials and so generally restricted to cement-based grouts. 3. May lead to significant hole deviation. 4. Collapsing strata will prevent effective grouting of whole stage, unless circuit grouting method can be deployed. 5. Weathered and/or highly variable strata problematical. 6. Packer may be difficult to seat in such conditions.	1. Grouted depth predetermined. 2. Hole may collapse before packer introduced or after grouting starts leading to stuck packers, and incomplete treatment. 3. Grout may escape upwards into (non-grouted) upper layers or the overlying dam, either by hydrofracture or bypassing packer. Smaller fissures may not then be treated efficiently at depth. 4. Artesian conditions may pose problems. 5. Weathered and/or highly variable strata problematical.



From: U.S. Army Corps of Engineers (1984)

Figure 50. Injection pressures used in U.S. grouting practice.

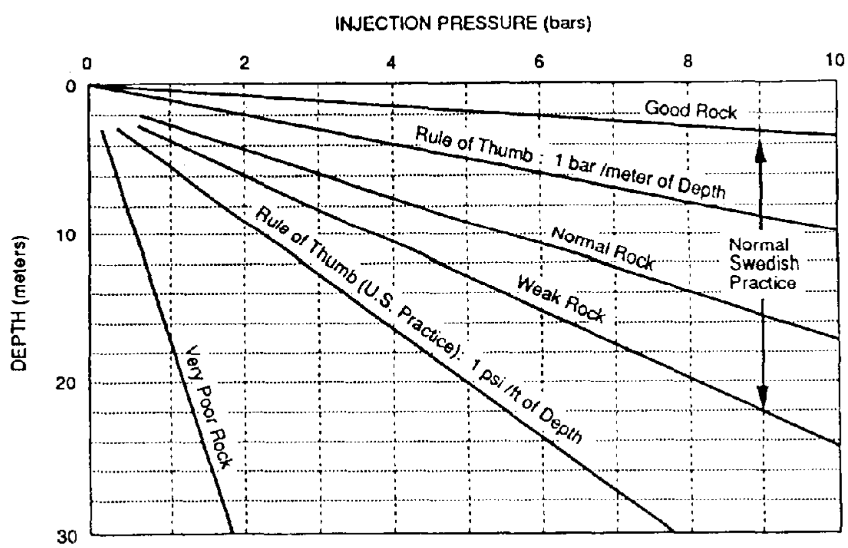


Figure 51. Injection pressures used in Swedish grouting practice.[5]

For rock grouting, as for all other grouting, pay items are listed separately. This approach, while not common in general construction, is usual for grouting and is the approach of choice of government agencies based on experience. Costs for routine instrumentation, though specified, are typically included in other items.

Because of uncertainties involved in rock grouting (i.e. the requirement for maximum flexibility to meet field conditions and the exploratory nature of grout programs), accurate estimates of quantities are extremely difficult. Many contracts contain language that reserves the right to increase or to eliminate any part of the drilling and grouting program without changing unit prices.

Grout and exploratory hole drilling are paid for on the basis of the linear meter of holes actually drilled and typically include the cost of washing, pressure testing, pressure washing, and removal of grout from holes for deeper stage drilling. In some contracts, pressure testing and washing are separate, hourly-based pay items because the inspecting agency on site might direct the time that these procedures are to continue.

Materials are paid for on the basis of weight of each component injected into the holes. Grout injection (or placement) is paid for on the basis of either the number of cubic meters of grout injected or by the hour.

b. Methods of Estimating

The volume and extent of work involved in a drilling and grouting program can only be approximated in advance of construction. Quantities are estimated for bidding purposes, but substantial variations are common. The contract specifications and bid items should be prepared so that the estimated quantities for each of the bid items may vary substantially without affecting unit prices. However, a concerted effort must be made to estimate the quantities of drilling and of grouting materials (e.g. grout take) that will be required. Test-grouting programs, boring evaluations, and unit-take estimates, are frequently used for estimating purposes.

Bid Items

The contract drawings and specifications should clearly indicate the drill hole spacings, sequencing, direction, maximum angle, maximum depths, and allowable deviation therefrom. The amount of drilling should be estimated on the basis of the job as planned and shown on the drawings, and the amount of drilling anticipated for each drilling item should be shown. The related quantities of water testing, grouting, materials and so on should also be carefully spelled out.

The following additional items should also be included in an estimate or bid schedule.

- Drilling Exploratory and Verification Holes. To determine the effectiveness of the grout curtain or portions thereof during grouting operations it will be necessary to drill such holes at key locations. Drilling of exploratory and verification holes will be measured for payment on the basis of linear meters of holes actually drilled.

- Pipe and Fittings. All pipe to be embedded in concrete or in rock through which holes will be drilled and grouted, and the fittings used in connection there with, should be covered by one pay item regardless of the different sizes used. The quantity should be estimated on the basis of the number of kilograms of pipe and fittings that will be required.

- Drilling Drain Holes - The drilling of drain holes should be covered by separate items for each hole size. Should both drilling in the open and from galleries be required on the same job, separate items for these conditions may be desired. The spacing and the depth of drain holes can ordinarily be predetermined with a greater degree of accuracy than can grout holes. The quantity for each item should be expressed in linear meters.

- Instrumentation - All instrumentation other than that integral to control or analyze the drilling and grouting data.

The type of rock to be treated and the purpose and target of the grouting program are major controls over the cost of any rock grouting project. To obtain information when preparing a bid package, it is recommended that input be sought from local Federal, State and private agencies, as well as from specialty contractors. As a general guide, it may be estimated that a grout curtain may cost $100-300 per square meter of curtain, including all drilling and grouting activities and materials.

4.6 CASE HISTORY(49)

The technical literature is rich with case histories of dam grouting projects, and to a lesser extent with tunnel grouting projects. This particular case history has been selected, however since it was the first U.S. application of microfine cement.

The Helms Pumped Water Storage Project, completed in 1982, is an 1,161 MW hydroelectric facility located in the Sierra Nevada Mountains in California. Underground features include 6,700 m of concrete lined pressure tunnel up to 8.2 m in diameter, 2,130 m of unlined access tunnels up to 9.5 m in diameter, three deep vertical shafts, one steeply sloping inclined shaft, and two major chambers more than 300 m underground (figure 52).

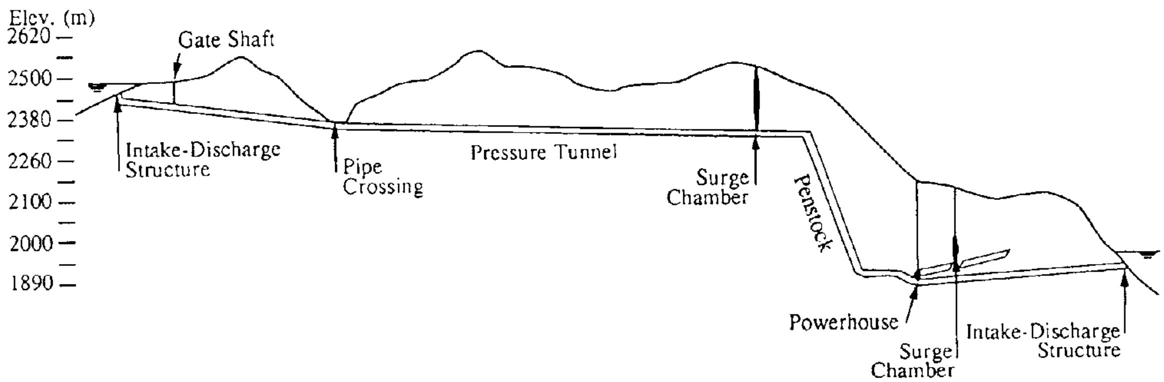


Figure 52. Underground features of the Helms Pumped Water Storage Plant.

During design, surface mapping and core drilling of this granitic terrain indicated that while tight, moderately spaced jointing in a cubic pattern was common throughout the rock mass, more severe discontinuities were rare. However, during excavation of the tailrace access and penstock access tunnels, a previously unidentified major shear zone was encountered (figure 53). The location of this shear zone was of particular concern because one of the tunnels it crossed was to carry high-pressure tunnel water, while the other was to remain dry. Groundwater seepage from the shear zone at each of these two tunnels suggested that the shear might act as a conduit for high-pressure tunnel water, allowing this water to pass around the concrete tunnel plug intended to contain it.

The original grouting program included contact and cutoff grouting for the concrete-lined pressure tunnel. This was expanded to include additional grouting in the vicinity of the tunnel plug and the shear zone. In addition, a series of piezometers and weep holes were installed in the rock downstream of the plug to monitor and relieve any high-pressure tunnel water that might enter and travel along the shear zone.

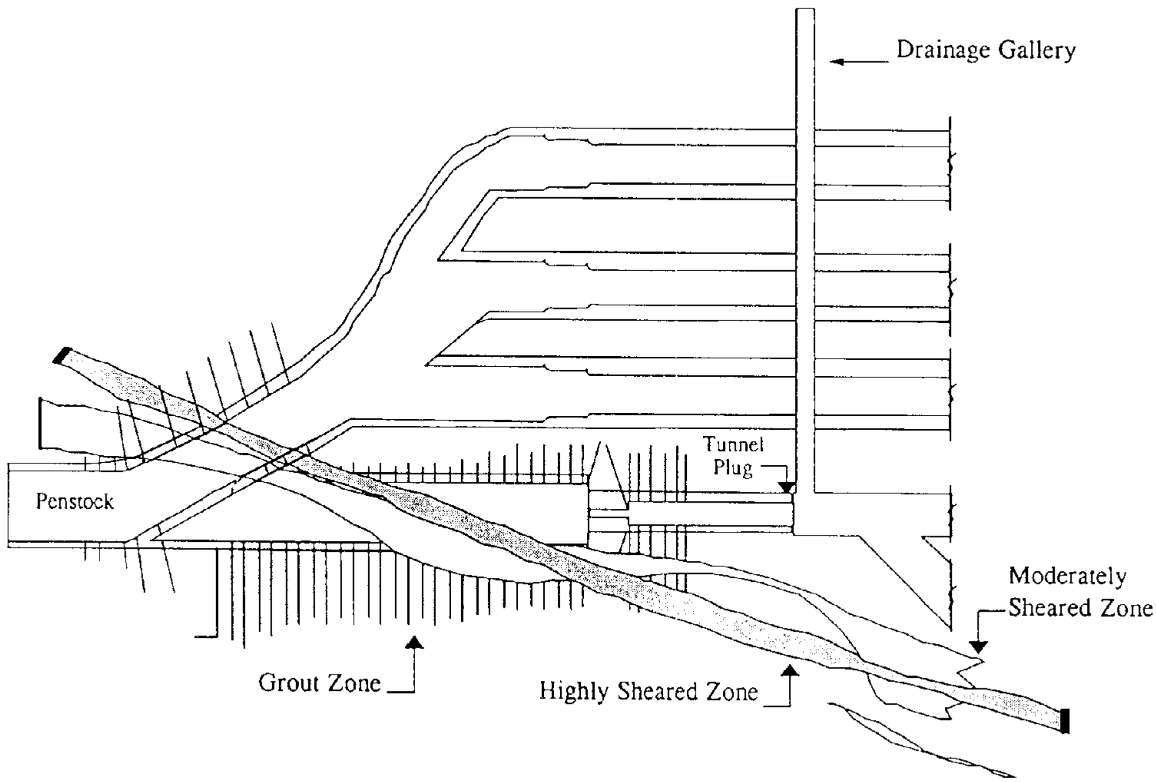


Figure 53. Plan view showing location of shear zone west of powerhouse station.

During the initial filling of the pressure tunnel, piezometric pressures in the rock downstream of the plug and groundwater seepage around the plug increased dramatically as water pressure in the tunnel increased. In addition, pressure and water flow in the shear zone showed a rapid response to changes in tunnel pressures and seepage elsewhere in the powerhouse complex. Of particular concern was that by the time water pressure in the tunnel had reached its maximum static pressure, the piezometric pressure in the rock exceeded the rock's minimum confining pressure in the direction of minimum principal stress. This condition which could lead to hydro-jacking or opening of existing joints, causing the water seepage rate to increase.

An extensive investigation confirmed that the zone could act as a high-volume conduit for both groundwater and high-pressure tunnel water. Of the remediation alternatives considered, pressure grouting was selected as the best method of creating a barrier around the pressure tunnel without closing the weep holes.

The objective of the grouting program was to limit groundwater pressures down stream of the plug to within acceptable limits and to reduce groundwater seepage to a minimum. The grouting contractor, who was responsible for grout mix design, determined that ultrafine cement grout would be needed for the deeper, higher pressure grouting because of its superior penetrating ability. Several grout mixes were developed for each cement type since different grouting circumstances were expected. The initial grout mixes were then modified as necessary based on experience gained as grouting progressed.

To protect the concrete tunnel lining and surficial rock from the higher grouting pressures, grouting was implemented in four stages: 0.3 m through 1.5 m, 1.5 m through 4.6 m, 4.6 m through 7.6 m, 7.6 m through 12.2 m. Rings of grout holes were drilled and stage-grouted on a primary, 12 m spacing along a tunnel section. Secondary, then tertiary grout rings split the primary spacing for a final spacing of 3 m. Drilling and grouting of each stage were completed before the next stage began.

Grouting data were continually monitored and evaluated. Also, periodic water-pressure tests were performed in test holes drilled into the grouted zone where takes had been high Information gained, along with pre-grouting water pressure testing of all holes, was used in conjunction with grout take information to modify the grouting program as needed.

When the pressure tunnel was refilled, piezometric pressure in the shear zone of the plug was shown to have decreased by as much as 20 percent, pressure at depth in the rock outside the shear zone had been satisfactorily stabilized, and total seepage from the rock downstream of the grouted zone was reduced by approximately 40 percent. These results were well within acceptable limits, confirming that the grouting program had met its objectives.

CHAPTER 5

BULK VOID FILLING AND SLABJACKING

Bulk void filling is employed in a large array of applications including limestone cavity infill, backfilling of old mineral workings, and repair of scour problems under bridges. Slabjacking is used to lift and relevel concrete slabs.

5.1 APPLICATIONS

a. Void Filling

Many regions of the United States are underlain by limestone rock formations. Due to its solubility in water, limestone tends to erode over time, thus forming cavities. These can potentially cause the ground above to collapse or sink if they migrate to the surface. These phenomena are known as sinkholes. Compaction type grouts can be pressure injected into these limestone cavities to seal the cavities and redensify the loosened overburden soil. The consistency of the grout prevents it from flowing through the network of caverns which often exist in limestone. In this way, localized filling and stabilization of an area can be accomplished and sinkholes can be prevented. Depending on the design of the grout, and the nature of the site, this approach can be adopted also in flowing water conditions.[(50)]

Drilling and grouting methods are commonly used to fill collapsed or abandoned mines to prevent surface subsidence, and this has major application in Ohio, Pennsylvania, West Virginia, Wyoming and Alabama in particular.[(51)]

An important application, especially for bridges, is the repair and/or prevention of scour. The stability of bridge abutments and pile foundations is commonly affected by scour, which occurs during normal movement of water around the bridge foundations and substructure, or during periods when the volume of water is abnormally high due to storms or ice melt. The voids that form underneath the piers and abutments may be corrected by the use of fabric bags filled with grout or concrete.

b. Slabjacking

Slabjacking is used to correct settlement damage associated with concrete slabs positioned over unstable ground material such as organic soils, compressive clays and silts, or materials subbase which have been washed or eroded away. This application is especially appropriate for highway maintenance activities.[52]

Slabjacking procedures include raising or leveling, under-slab void filling (no raising), grouting slab joints, and asphalt subsealing. Most slabjacking uses a suite of cementitious grouts, supplemented with bentonite, sand, ash or other fillers, as dictated by local preference and the project conditions and goals. Certain proprietary methods use expanding chemical foams to create uplift pressures. Best results (results without cracking) are obtained when the slabjacking is uniformly and gradually conducted. Slabjacking can also be used to "pump" expansion joints which have sunk below the adjoining section.

A 1977 study, *Slabjacking-State-of-the-Art* summarized the various slabjacking practices then employed by State Transportation agencies as follows:[57]

1. Slabjacking (raising or leveling) - 25 states.
2. Under-slab void filling - 17 states.
3. Grouting slab joints - 6 states.
4. Subsealing (hot asphalt) - 3 states.
5. Filling voids prior to overlay - 6 states.

5.2 ADVANTAGES/DISADVANTAGES

a. Void Filling

When a void or cavity affects surface structures, grouting is usually an economical method of solving the problem and on many occasions is the only viable solution. The strength of the grout can be tailored to suit the in-situ condition. Similar to other forms of rock grouting, the drilling and grouting can be considered an extension of the exploration program while also remediating the problem. One problem is the difficulty of completely filling the void. Another problem is containing the grout within the zone to be stabilized, although the use of low slump grout "barriers", accelerated grouts, and grout filled fabric forms have been used to minimize this problem as illustrated in figure 54.

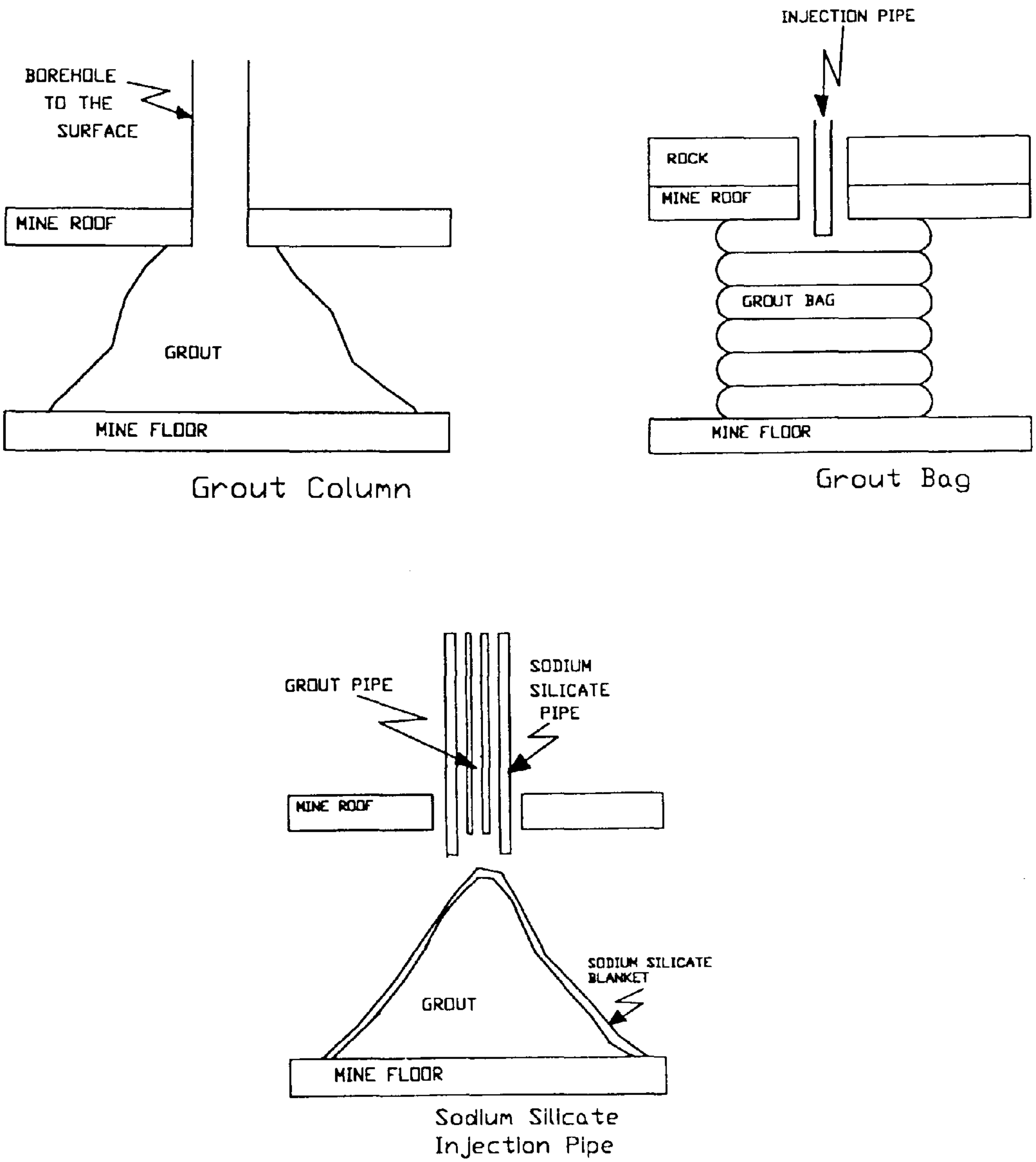


Figure 54. Void filling methods.

b. Slabjacking

The advantages of slabjacking include the following:

- It is frequently the most economical repair method.
- It is usually faster than other solutions, especially compared to removal and replacement.
- It can be planned so that there is little disruption to the existing facility and can be performed at times of light or no traffic.
- The equipment needed to perform the slabjacking operation can be removed from the repair location, providing for maximum accessibility.
- Increased load capacity of the slab is provided.
- The useful life of the concrete surface is extended.
- A smoother riding surface is established.

Slabjacking has the following disadvantages:

- Cracks and visible distress already present may tend to open up when the slab is treated unless great care is taken with the process.
- Slabjacking may not be cost-effective on small projects.
- Slabjacking does not address the original cause of the settlement.

5.3 FEASIBILITY EVALUATION

a. Scour Repair

A diving inspection of a scour problem is frequently necessary to assess the most expedient and economical method of repair. High-strength flexible fabric forms have demonstrated their capability for rapidly, economically and permanently repairing scour problems. The fabric forms can be manufactured on site to meet actual site conditions. This solution is designed to serve both as future scour protection and as a form to allow the structural grout or concrete to be placed in a void under the foundation in a controlled manner. Also, grout contained in these forms will not add to the dead load of the bridge piers by bonding to the piles. This fact is important because the substructure unit must be able to hold the additional dead load if the concrete or grout bonds to the piers. The main limitation of the forms is that their height is normally one-half of their width. Experienced divers should be employed, as these fabric bags, when being inflated with grout, contain a fluid mass which can roll and trap a diver. When multi-layered tubes are utilized, they are normally doweled together with the dowel connectors placed in the fresh grout of the lower tubes and extended into the upper tubes prior to inflation.

b. Void Filling

As in all grouting operations, many sources of information have to be studied before the feasibility of a solution can be established. The information studied for void filling should include historical as-built data on mine and tunnel projects. These can often be supplemented by visual surficial, or underground assessments, where man-access is practical and safe. Assessment of karstic terrains is often more difficult, and will involve intensive exploration drilling, usually supplemented by a variety of geophysical (eg. GPR, resistivity), and hydrological tests.

As a basis for design therefore, the lateral and vertical extent of the voids or collapsed zones must be determined, together with an indication of the ground water regime, and in particular if the water is moving, and if so, at what velocity and rate.

In general, it is not uncommon to find such projects involving the drilling of several hundreds of holes, to depths of over 300 feet, and the injection of varieties of grouts formulated from hundreds of thousands of tons of materials.

c. Slabjacking

When a slab or structure has settled differentially, a cost analysis is key in determining whether to replace the slab and correct the cause of the problem or to jack the slab back to its original elevation and repeat this process periodically. Slabjacking is typically not appropriate where the cracking is severe. Local contractors can be contacted to provide budget estimates and feasibility studies.

5.4 DESIGN OF BULK FILLING AND SLABJACKING

a. Problem Definition

Void Filling

Where the feature is completely void, good grouting practices can be used to provide full filling. However, if clay or other erodible material is present as infill, then it is best to remove as much of this material as possible prior to grouting. Removal can be achieved by flushing with air and/or water and/or dispersant.

It is important to realize that the extent of a cavity is unknown after penetration by just one grout hole. When a cavity is encountered in drilling, the hole is normally stage grouted. Sometimes, it may be necessary to use intermittent grouting, which is the process of injecting grout in the hole and then waiting several hours before injecting more grout. In practice, the maximum quantity of grout to be injected varies from about 300 sacks to over 1,000 sacks per injection period. A limit may also placed on the maximum amount of grout to be injected into a single hole. This practice differs from that permissible in fissure grouting practice.

When grout injection refusal is reached, it is assumed that grout has filled at least the portion of the cavity penetrated by the grout hole. Additional grout holes are then drilled and grouted until the desired results are achieved in a split spacing sequence. If pressures fail to build up or the cavity is too large to grout in this manner, grouting should continue with a grout curtain placed to control the flow of grout from the cavity. Additional exploration, consultation, evaluation, and design of treatment can then take place without delaying the contractor. These measures may call for specialized grouting procedures or materials such as foaming agents or accelerators, positive cutoff diaphragms or formed concrete wall, additional excavation, or some other solution. Hole spacings and locations will be dictated by the site conditions but holes on a final grid spacing of 3 m or less are not unusual.

Slabjacking

Prior to undertaking any slabjacking program, the underlying cause of the problem and the desired end results must be determined. If slabjacking is used for settlement re-leveling, future leveling may be required. If the roadway pavement that is to be stabilized is to receive an overlay, virtually no lifting may be required. Regardless of the cause of the problem, the engineer should accurately specify the necessary performance requirements and tolerances for the project.

Another consideration is the appearance of the finished surface. Most slabs that have settled contain at least some cracks. Although slabjacking can be performed without creating new cracks, those cracks already existing will be visible.

Slabs restored by slabjacking will contain patched injection holes usually on a grid of 1.5. to 1.8 m. Therefore, the surface finish conditions should be considered in advance of the work. These factors will vary depending on the affected facility. While minor defects may be tolerable on a highway, they will not be acceptable on a tennis court.

b. Performance Evaluation

Void Filling

Borehole cameras are available that can be placed in adjacent drill holes to observe and verify that the injection of grout is satisfying project requirements. Instrumentation can also be specified to monitor heave, settlement, etc. during the grouting program, while the close analysis of grout volumes and pressures attained during each phase of grouting remains a classic performance monitoring technique.

Scour Repair

Divers are normally required for repair of scour problems. However, the turbidity of the water can cause misleading interpretation as to the status of the problem. The use of borehole or underwater cameras can also be valuable in evaluating this problem.

Slabjacking

The objectives of slabjacking are to fill voids and raise the slab to its approximate original elevation without causing additional damage to the slab. Instrumentation as simple as a string line can ascertain this objective, although the use of lasers is more accurate.

5.5 CONSTRUCTION AND MATERIALS

a. Equipment Requirements

Void Filling

Because void filling often parallels the grouting techniques discussed in previous sections, much of the same standard equipment is utilized, as described in chapter 4. Selection of equipment is dependent on project requirements, but will typically be of large scale and capacity.

Slabjacking

Slabjacking equipment includes grout mixers, agitators, grout pumps, and high-pressure hoses. A grout mixer equipped with suitable water measuring devices is needed to produce uniformly proportioned grout and to control the grout consistency. Pumps capable of developing pressures of 2,800 kPa or less and displacement rates of 0.007 m^3/min are sufficient. However, it is preferred that the pump develop pressures to 10,350 kPa and displacement rates varying from 0.003 m^3/min to 0.056 m^3/min.

b. Materials

Void Filling

The materials used in a void filling grouting operation can vary from noncementitious waste materials to high-strength, low-slump concrete depending on the purpose and intent of the project. Void filling usually encompasses one or more of the other grouting techniques and so the materials utilized in void filling vary considerably. The manual sections corresponding to these different grouting techniques should be referenced when beginning a void-filling operation.

When filling scoured zones with concrete filled tubes or bags, a fine aggregate concrete (structural grout) is recommended. The typical range of mix proportions is shown in table 6.

Table 6. Typical range of material mix proportions for void filling applications using concrete bags.

MATERIAL	MIX PROPORTIONS (kg/m^3)
Cement	600-750
Fly Ash	180-220
Sand	2150-2300
Water	525-600 (as required)

Slabjacking

Most slabjacking operations can be successfully completed using a grout composed of portland cement, fine sand, and water although bentonite and chemical admixtures may be used to provide appropriate rheological properties. Cement content varying from 5 percent to 10 percent, depending upon the sand gradation and admixtures, will be sufficient to provide a grout strength in excess of 480 Pa. Where higher strengths are needed, higher proportions of cement can be used. Water content should be adjusted to provide the necessary consistency. Ideally, sand material should be well graded with 100 percent passing a 2.38 mm sieve with not more than 20 percent finer than 50 μ. Calcium chloride or high early strength cement can be used to accelerate the set, and admixtures that can control the shrinkage or expansion can also be added. Where exceptionally high strengths are needed or excessively coarse sands must be used, admixtures are generally not used. In these cases, pozzolan (15 percent-50 percent of the weight of the cement) will improve the pumpability of the grout.

5.6 COST DATA

Void Filling

The cost of void filling projects involves (1) mobilization and demobilization, (2) drilling (production and exploratory), (3) flushing and water testing, (4) mixing and injecting grouts, and (5) materials. The mobilization/demobilization cost will vary based on the complexity and number of drill rigs and grout plants required. The mobilization of a single drill and grout plant should be under $15,000. In most grouting projects to fill voids, overburden materials and rock must be penetrated to reach void elevations. Normally, a primary, secondary, and sometimes tertiary hole spacing is utilized. The primary grouthole grid pattern may range from 3 to 30 m on center. The diameter of the drill hole normally ranges from 76 mm to 203 mm and cost starts at $25 per linear meter. The cost for supplying, mixing and injecting the grout normally ranges between $75 and $200 per cubic meter.

A review of costs for bridge scour repair using concrete fabric forms from 1968 to 1976 in Pennsylvania indicates a range of $300 to $1,000 per cubic meter.[52A]

Slabjacking

Due to the extra effort involved in delicately raising a slab, the unit cost is normally higher than for only filling voids beneath slabs. For estimating purposes, a cost of $300/cubic meter of grout injected is a good starting point. Slabjacking using polyurethane may be estimated at $70 to $100 per square meter of slab raised up to 50 mm.

5.7 CASE HISTORIES

Case 1 - Mined Out Areas; Rock Springs, WY

This section of Sweetwater County, Wyoming has a number of old, abandoned coal mines. Over the course of time, surface holes and structure settlements had occurred due to mine subsidence. To mitigate this problem, the State of Wyoming initiated a program of subsurface repair and specified zero slump compaction grouting to construct pillars in the 1.8 to 2.4 meter-deep voids.

The grouting contractor performed a test program to verify the feasibility of the grouting approach. Pillars, 2.4 m in diameter, were constructed on 6 m centers using a zero slump grout. For every void, a telemetry sounding device was used to measure the depth of the hole prior to grouting. Core samples were taken to verify that the design parameters had been met. Subsequently, a number of contracts were completed in the area based on the success of the test program.

Case 2 - Limestone Cavities Filled at the Southeast Paper Manufacturing, Dublin, GA

Plans were made to expand an existing paper mill founded on a limestone formation that had undergone previous sinkhole activity. It was determined that, because of future static and dynamic loads, a failure of the surrounding area was possible. Geotechnical tests taken at the site revealed that large voids existed in the limestone formation 15 m below the ground surface. The site had not evidenced damage at the time of the trials.

Slurry grouting was selected as the preferred grouting technique. Grout holes were placed on a 6 m by 6 m grid pattern, utilizing grout consisting of sand, cement, and bentonite with a slump of 200 mm.

Significant grout takes in adjacent holes prompted the engineers to drill and grout secondary holes. A total of 2,294 m^3 of grout was injected through 4,880 m of drilled holes.

Initially, grout was injected into the primary holes that filled voids ranging from 1 to 4.5 m in depth. However, due to secondary drilling, adjacent voids were filled and the grout fully filled the problem area making the construction of the paper mill feasible.

Case 3 - Use of Fabric Tubes to Rectify Scour; Northumberland County, PA[52A]

While performing a bridge inspection in the fall of 1968, District 3-0, Pennsylvania Department of Transportation, encountered a void beneath Pier 16 of L.R. 25, Northumberland County. This 28-span through-girder bridge crosses the Susquehanna River between Sunbury and Shamokin Dam, PA. A recreational dam was planned immediately downstream from this bridge, which would raise the water level to a greater depth and would change the flow characteristics of the river. Because of these facts and the seriousness of the underscour, it was elected to repair the problem immediately.

The river at this point is approximately 762 m across, which would make the cost of an access road and cofferdam extremely expensive. The pier rests on a rock bottom which would complicate the installation of a sheet pile cofferdam. Therefore, PennDOT elected to try a new technique of placing concrete underwater by the utilization of a fabric form.

A nylon tube was designed to fit into the void caused by scour between the top of the rock and the bottom of the pier foundation. This tube was pressure-filled with pumped fine-grain concrete and extended partially into the void and partly outside the pier. Prior to inflation, pipes were placed into the void beneath the pier and, after the tube of nylon was inflated with the pumped concrete, the same concrete mixture was then injected into the void space behind the tube as shown in figure 55. Sufficient pipes were placed so that water could be vented out from the void, thus ensuring complete filling of the void.

In June of 1972, a considerable number of bridges were destroyed in the Susquehanna Valley of central Pennsylvania as a result of Hurricane Agnes. Subsequent investigations by PennDOT indicated that many other bridges had experienced scour problems beneath their foundations. In the course of the bridge-damage inspection, the treated bridge was re-investigated, and it was found that the tube of concrete was in place and no further scour was experienced at this pier. However, eight other piers of this structure had been damaged. These piers and other piers and abutments were also then repaired by the fabric form techniques. During Hurricane Eloise in 1975, flows once again exceeded the one-in-a-hundred-year prediction. Additional diving inspections showed that structures repaired by the fabric form technique had experienced no further major scour problems.

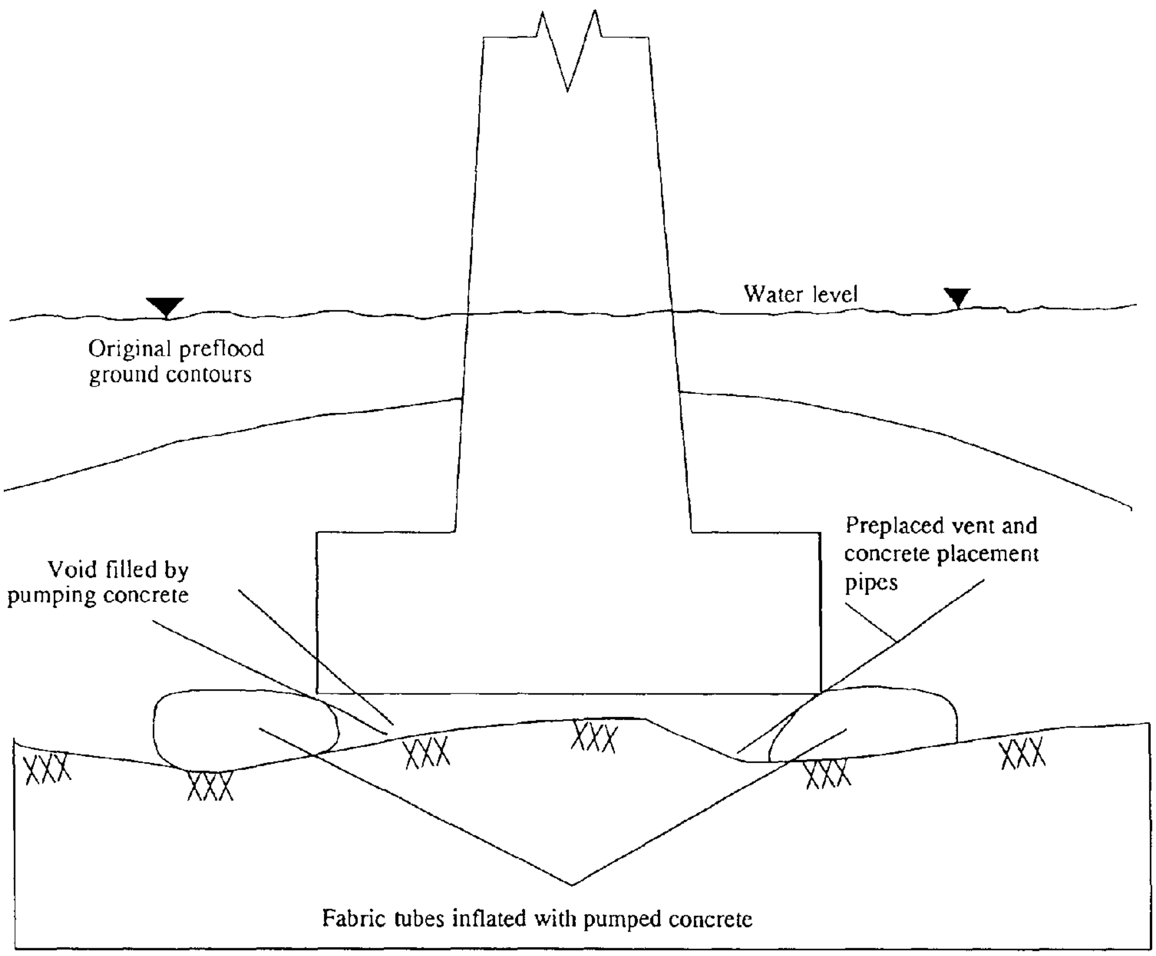


Figure 55. Void space filled by concrete-filled fabric tubes.

CHAPTER 6

BIDDING METHODS AND CONSTRUCTION SPECIFICATIONS

Construction specifications for grouting operations should be tailored to the specific project objectives. Caution should be exercised when attempting to use a preexisting "standard" specification without modification. It is common practice to "lift" sections from specifications from other projects, and to use them as a "boiler plate" to piece together a new specification. This practice is reasonable if the engineer doing so has verified that the meaning and the significance of requirements or provisions contained in existing specifications are clear, that they functioned as intended on the previous project, are equally applicable for the proposed project and are not mutually contradictory. Conversely, if not very judiciously employed, use of "boiler plate" specifications can cause problems and can help perpetuate the use of outmoded procedures, and/or inappropriate material, and may directly contribute to contractual disputes.

The guide specifications provided in this chapter should never be used directly; they should be adapted to the specific conditions and needs of a particular project.

6.1 BIDDING METHODS AND BID ITEMS

For grouting projects, it is highly desirable to establish the qualifications of potential bidders prior to the solicitation for bids. This process should go well beyond identifying contractors with experience in drilling holes and in mixing and pumping grout. Considerations should include verification of the prospective bidders' previously demonstrated ability to mobilize and utilize sufficient, appropriate equipment and competent, experienced personnel to satisfactorily accomplish the proposed work within the desired time frame. The prospective bidders should be required to submit the resumes of their key personnel, including their proposed project engineer and proposed superintendent(s). These resumes should specifically indicate directly applicable experience. The names and phone numbers of owners' representatives should be provided so they can be contacted for information regarding their satisfaction with each contractor's personnel and services.

Rock and soil grouting projects are specialty construction activities usually involving sophisticated techniques, equipment and materials. Specialty geotechnical contractors are therefore required, as opposed to general contractors without relevant experience.

Evaluation of the effectiveness of the grouting program must not only be constant but should also be a joint effort between design, construction and supervisory personnel. If problems develop, the response must be expeditious. Flexibility must therefore also be a feature of any grouting program, and this should be addressed in the contract documents.

a. Types of Contract

Grouting may be a part of a general construction contract or may be performed under a separate contract. The type of procedure to employ is dependent upon the project complexity and completion schedule, existing economic conditions, technical and manpower considerations, organizational structure, and workload.

Performing a grouting program under the general construction contract eliminates contractual difficulties that might arise from interference between these operations and other construction activities. Most general contractors, however, do not have grouting equipment, and the grouting is sublet to subcontractors specializing in this type of work, except in the case of tunnel grouting where the tendency is still to perform all work "in house".

Accomplishing a grouting program under a separate contract allows the grouting specialist to be a prime contractor.

Grouting can be bid under a Performance or Method (Prescriptive) contract, as described in chapter 2.

b. Methods of Estimating Grout Takes

- Test Grouting. For medium and large projects, probably the most reliable method for estimating grout take is to conduct a test grouting program, preferably during the design stage. The site chosen for testing should be geologically characteristic of what was found during subsurface exploration, and the means, methods and materials must be substantially those envisaged for the production work.

- Evaluation of Subsurface Information. The evaluation of the samples from the subsurface program, as well as the results of water pressure tests and other tests, is a fundamental part of the initial stages of preparing a grouting estimate. However, care should be exercised on grounds of site variability, and technical complexity.

- "Unit Take" Estimates. A method frequently used during preparation of detailed estimates for drilling and grouting programs is called the "unit take." In this procedure, the area to be grouted is divided into horizontal reaches and vertical zones of varying properties based on site geology and in situ test results. Estimates are made of the number of primary and split-spaced holes required to complete each area and zone.

- Experience. Often local knowledge of contractors or engineers is invaluable in providing a "reality check" on quantities derived by other methods.

c. Bid Items

Experience indicates that the following items should be included in any estimate or bid schedule for a drilling and grouting program.

Mobilization and Demobilization, Lump Sum

Drilling and grouting equipment must be assembled at the job site before a grouting program can be started and must be removed from the site when the work is completed, regardless of the amount of work actually performed. A separate pay item for these operations, therefore, should be included in the specifications, and the contractor will be guaranteed payment whether work under the other items of the program is performed or not.

Environmental Protection, Lump Sum

A separate pay item may be included in the specifications. Environment protection is defined as the retention of the environment in its natural state to the greatest possible extent during project construction.

Drilling Grout Holes, Linear Meter Rock, Soil

A minimum diameter hole is generally specified. If different diameter holes are required by the contract, separate pay items should be provided. Separate pay items may also be warranted for the various depths or angles or where some of the drilling is to be done under special conditions, such as from a gallery or tunnel. If it becomes necessary through no fault of the contractor to drill the grout from a hole after set, a special payment provision for redrilling should be provided (typically 50% of the rock drilling rate).

Pressure Washing and Pressure Testing

Preliminary washing of the grout hole usually is included for payment as a part of the drilling operations, and a separate pay item is not necessary. Pressure washing and testing are essential parts of the grouting program and therefore should be paid for as a separate item. Quantities of pressure washing and pressure testing ordinarily are measured for payment purposes in terms of units of time required to do the work. Pressure washing and pressure testing are closely related and the operations performed are similar; therefore, payments for both operations may be combined in one pay item. Although the extent of pressure washing will depend on the conditions actually encountered, an approximation of the amount that will be required, as well as the amount of pressure testing expected to be done, should be made for inclusion in the estimate.

Grout Placement, by Volume or Pump Hour

The pay item for placing grout should cover the labor, the use of equipment, and the necessary supplies (other than grouting materials) required to mix and to inject the grout into the holes. Placing grout is frequently paid for by the volume of mixed grout and/or by the pump hour. An estimate of the quantity of grout must be made even though the actual amount is not known in advance. Payment for grout injection by the hour may be more appropriate in certain cases and would include labor and use of equipment to inject the grout into the holes.

Connections to Grout Holes, Lump Sum or per Connection

The labor required to hook up to a grout hole is independent of the effort involved in placing grout, and a separate payment may be desirable for each hookup or connection. The payment may consist of a fixed or bid price per grout hookup or connection.

Grout Materials, by Volume or Weight

Separate pay items should be established for each of the grout materials (except water) anticipated or planned to be used. The estimated quantity of each, expressed by volume or weight, should be derived from past experience, knowledge of the geologic conditions, and from test grouting, if performed. Clear distinction must be drawn with respect to the items being paid for under "Grout Placement" (above). The volume of grout placement must be consistent with the weights of the various grout materials.

6.2 SOIL GROUTING SPECIFICATIONS

a. Chemical Grouting (Permeation Grouting)

The specialty contractor's expertise is especially needed for the following steps: (1) development of a grout pipe layout scheme and installation of sleeve-port grout pipes in a precise pattern; (2) development of a rational injection sequence plan with proper allocation of grout volumes to the various grout ports; (3) proper operation of the grout mixing and injection system in harmony with the actual ground response; (4) continuous recording (preferably automatic), graphical display and analysis of the injection data; and (5) quality assurance acceptance testing. The integration of the technical and mechanical skills required by the above is a complex and ongoing process that is continuously adjusted during the construction process. This situation may preclude the design engineer from directing the exact details of the work.

The following Guide Specifications for chemical permeation grouting were prepared to enable the designer to specify the appropriate performance requirements and construction monitoring to ensure a successful project. The specifications also permit the specialty contractor the freedom necessary to accomplish the five-part outline addressed above. The work, if performed as outlined, can be easily monitored by project construction management staff on the job, and performance problems will be quickly highlighted and more easily corrected. These specifications define the intent and extent of the work, establish specialty contractor qualifications, set criteria for grout selection, describe acceptable pumping equipment types and operating procedures, specify grout pipes, define injection procedures and quality control, and establish the basis for acceptance and payment.

Guide Specifications
(Chemical Grouting)

Scope of Work

The work covered by this specification consists of furnishing all supervision, labor, materials and equipment necessary to perform the grouting as hereinafter specified and as outlined on the contract drawings.

Intent

The purposes of the chemical grouting program are either (1) to increase the strength and stiffness of the in-situ soils in order to reduce surface settlements due to subsequent excavation operations to less than the values shown on the contract drawings, or (2) to sufficiently decrease the permeability of the affected soils below the existing groundwater table so as to permit excavation without dewatering (or to a maximum target inflow rate) or, (3) both. *(Designer to specify intent)*

Structural Chemical Grouting

The areas and depths specified for structural chemical grouting are identified on the plans. The chemical grouting should be performed in such a way as to produce a continuous mass of structural chemically grouted soil as shown on the drawings. Some treatment areas will require combined structural and water control chemical grouting effects.

Waterproof Chemical Grouting

In the zones specified for waterproof chemical grouting, the chemical grouting should be performed in such a way as to produce a continuous wall of chemically grouted soil below the water table. Acting as a barrier, this wall will prevent or reduce the flow of water beyond to the target value.

Qualifications

The work must be performed by a contractor experienced in chemical grouting. The contractor should have completed at least three chemical grouting projects of similar scope and purpose. The subcontractor should also have experience using the specified mixing procedure, automatic recording equipment, and types of chemical grout. The contractor should also establish to the Engineer's satisfaction that the on-the-job supervision of all chemical grouting is under the direction of an engineer with at least three years actual on-the-job supervision in similar applications, assisted by an experienced chemical grouting foreman on each grouting shift. Proof of experience requirements should be submitted with the work plan, as described below.

Installation of Grout Pipes

Grout pipes may be installed horizontally, inclined, or vertically to obtain the specified minimum grout coverage, with a maximum average spacing between adjacent grout pipes of 1.5 m. The grout pipes should be of the sleeve-port type, with grout ports at 0.3 to 0.5 m depth intervals covered by expandable rubber sleeves *(Engineer to specify)*. The sleeve-port pipes should be installed such that a cement bentonite grout fills the annulus of the borehole. An internal double packer is to be used to inject grout at each specific sleeve-port.

Work Plan

At least 30 days prior to the start of the drilling work, the contractor must submit a detailed Chemical Grouting Work Plan, specifying the chemical grout to be used, grout-hole and grout-port locations, grout-pipe installation procedures, grouting equipment, injection procedures and sequences, recording equipment, data reporting methods, and schedules for the review and approval of the engineer. The plan must show the basis for establishing grout target volumes at each primary and secondary grout port.

Grout Mixing Method

[Note: Some contractors may favor other methods such as batch mixing: they <u>*must*</u> *be permitted to offer this alternative as long as the project requirements can be met.]*

The method of injection for chemical grouting should be the continuous mixing method, with the proper amounts of sodium silicate base material, water, reactant, and accelerator automatically proportioned and continuously supplied at proper flow rates and pressures. The batch material and the water-accelerator-catalyst solution should pass through parallel separate hoses to a suitable baffling chamber near the top of the hole. A sampling clock, to allow frequent gel-time checks, should be placed after the baffling chamber. Suitable check valves should be placed in the grout lines at the proper locations to prevent backflow.

Injection Procedures

Using double packers, chemical grouts should be injected into the design zones through grout ports in the sleeve pipes. The net grouting pressure for any one pipe should not be more than 29 kPa per meter of depth, unless acoustic monitoring or similar in hole geophysical testing is to be performed in adjacent boreholes to detect hydraulic fracturing, in which case net pressures may be increased as desired up to 74 kPa per meter of depth. Detection of excessive hydraulic fracturing will require reduction of injection pressure. Surface elevation monitoring will be carried out continuously during grouting. Injection procedures will be adjusted as needed to prevent excessive surface heave. Temporary high injection pressures are permitted to crack open sleeve-ports, but these pressures should not be permitted for longer than 1 minute duration. In any event, the rate of injection into any port should not exceed 40 liters per minute.

Gel Times

All grouts should have a gel time between 5 and 50 minutes, with most grout having gel times in the range of 15 to 40 minutes. Samples should be obtained for gel-time checks at least one for every half hour of pumping or for every 2,000 liters of grout, whichever is more frequent. Gel samples should be properly labeled and stored until the completion of the project.

Quality Assurance

Accurate and timely records of all chemical grouting should be kept by the grouting subcontractor and submitted to the engineer. These records should include, but not be limited to, grout mix, gel time, injection date and time, injection pressure and rate, injection volumes, and exact injection location. In addition, these data should be displayed in an acceptable chart type format that facilitates rapid visual evaluation of the results of the work. This display should be updated daily. The engineer should review the daily work sheets to ensure that the injection plan and quantities conform to expectations from the subsurface conditions.

Prior to the commencement of work through or near a grouted zone, the contractor should demonstrate, using either soil sampling methods or geophysical methods such as radar, acoustic velocity measurements, or other means satisfactory to the engineer, that the grouting zones have been thoroughly impregnated and stabilized with chemical grout. Work through or near grouted areas should not commence until the chemical grouting work has been completed and accepted by the engineer. *[Note: Chemically grouted soils do not usually lend themselves readily to conventional coring methods as a verification technique.]*

Basis of Payment/Units of Measurement

Payment should be made for the work based on the following unit prices:

1. Mobilization/Demobilization. The cost of assembling all plant, personnel, and equipment at the site prior to initiating the grouting program and the cost of removing the same upon completion will be paid in the contract lump-sum item for "Mobilization/Demobilization."

2. Placement of Grout Pipes. Grout pipe placement should be measured for payment on the basis of the number of meters of sleeve-port grout pipe properly placed, measured from the ground surface to the bottom of the pipe, and including the bentonite-cement sleeve grout.

3. Injection of Grout. Injection of grout should be measured for payment by the liter of liquid grout properly mixed and injected.

4. Grout components (such as sodium silicate, reagent, accelerator) should be paid for by liter supplied (excluding water).

5. Quality Control and Testing. Quality control during grouting will be paid under the grout items per liter and will not be paid separately.

6. Special instrumentation for performance monitoring and in situ verification can be listed separately, or deemed included in the other rates.

b. Compaction Grouting

As discussed in chapter 3, Compaction Grouting has many applications. This guide specification applies primarily to compaction grouting for structural leveling. Compaction grouting for other applications may require adjustment to, for example, grout mixes and refusal criteria, on a project specific basis.

Guide Specification
Compaction Grouting

Scope of Work

In connection with the compaction grouting as shown on the drawings, the grouting contractor should provide all labor, materials and equipment to accomplish the following items of work:

- Submit detailed compaction grouting and associated ground movement monitoring program.

- Install ground movement monitoring system.

- Install and remove grout pipes.

- Implement compaction grouting program in coordination with instrumentation monitoring and construction operations and under supervision of an engineer experienced in compaction grouting for settlement control.

- It is the contractor's responsibility to design the systems to ensure that structural settlements do not exceed the maximum allowable values indicated in the contract documents.

Contractors Qualifications

The firm performing the compaction grouting shall submit evidence of experience and the successful completion of five comparable projects within the last five years.

Personnel

The on-site representative of the contractor should have at least five years experience supervising this type of work. He should also have at least three years employment with the contracting firm performing the work to assure full and competent knowledge of the contractors equipment, personnel, procedures, etc. In addition the on-site representative of the contractor must have sufficient experience and knowledge in performing/supervising this type of work to evaluate incoming data, troubleshoot the wide variety of problems/situations inherent to the work, and communicate with the Owner regarding job status.

Equipment

The equipment used to mix, pump and place grout shall be specifically designed for this purpose. Because of the high pressures involved with a properly designed and implemented Compaction Grouting Program, all equipment used shall be able to operate well above standard requirements.

1. The grout-mixing system shall be capable of thoroughly mixing a true low slump grout (averaging 20 mm or less but in no case exceed 38 mm as measured by ASTM C-143-78). The mixing unit shall be capable of precisely measuring and mixing the aggregates, cement and water. Calibration of the mechanical metering system shall be performed at the beginning of every job, under the Engineer supervision to insure proper ratios are being delivered and mixed. The Engineer shall have the right to check the mix design and calibration at his discretion. Alternatively, grout of appropriate composition, and uniformity, can be supplied in readimix trucks.

2. The grout pump shall be a positive displacement, variable speed type. The pump must have the capability of injecting the low-slump grout at a pressure of up to 6.9 MPa as measured at the point of injection, and a pressure of up to 13.8 MPa at the pump. The pump shall have a minimum injection capacity of 4 liters/minute and a maximum up to 140 liters/minute.

3. The mixing system and grout pump shall be designed in order to provide continuous flow of the grout mixture without interruption due to inadequate batching capacity.

4. All pressure gauges shall be adequately protected in order to provide accurate pressure readings on a continuous basis and shall be calibrated by a reputable instrumentation firm at least twice every year, or as required by the Engineer.

5. There shall be a reliable, mechanical or electronic means of measuring the quantity of grout pumped in every stage.

6. A remote system allowing on-off operation of the grout pump from the injection point shall be provided and maintained in good working order.

7. Drilling equipment shall be capable of drilling or driving grout probes (wet or dry) to depths determined by the Engineer without destructive measures for access.

8. Grout probes shall have a minimum inside diameter of 50 mm. The casing shall be "flush joint" both inside and outside (no exterior couplings) and capable of pull-back pressures of 30 tons pull without pulling apart. The casing shall be capable of unobstructed flow out the tip or a minimum 50 mm diameter unobstructed extrusion flow.

9. The casing retrieval system shall have appropriate pulling capacity to withdraw the grout casing in controlled minimum 0.3 m stages. Any holes or casing lost during the course of the project due to the inability of the contractor to pull the casing shall be replaced at the contractors expense.

10. Length of the casings and casing retrieval system shall accommodate the overhead constraints with minimal or no destructive measures to the structures.

Materials

1. A grout mix with high internal friction shall be used, averaging 19 mm or less but in no case to exceed 38 mm (as determined by ASTM C-143-78). Slump tests may be performed twice daily or more frequently as desired by the Engineer. *Grout Strength requirements may vary from project to project so it shall be the Engineer's responsibility to determine specified strength and the contractors responsibility to achieve it.* Also, as noted above, slump may be varied depending on site specific factors.

2. Use of natural or artificial clay type substances as additives to improve flow or pumpability of the low-mobility grout shall not be allowed. The Plasticity Index of materials passing the 200 sieve shall be less than 20.

3. Cement; (Type to be specified) The cement shall meet ASTM specification C-150 and be stored in an approved manner. Amount required to meet strength requirement shall be determined by Contractor.

4. Water; As needed to meet slump requirements. Water shall be clean, potable, and free of contamination.

5. Fine Aggregate; The material shall be a silty sand with less than 25% passing the 200 sieve.

6. Additives; no additives shall be allowed unless approved by owners representative or engineer.

An adequate inventory of grout material shall be stockpiled in order to eliminate unnecessary delays in the grouting operation due to material shortages.

Monitoring and Records

1. Initial survey readings shall be performed by the Owner in order to determine existing elevations of the *(describe structure)* being grouted. The Engineer shall determine the tolerances to which the structure shall be releveled and upon completion of grouting operations, additional readings shall be performed to provide a permanent record.

2. The contractor shall be responsible for monitoring of structures during the Compaction Grouting operation. The monitoring shall detect movements greater than 3 mm. These systems shall include, but not be limited to:

   a. Multiple manometer survey systems, for both determination of current relative elevations and detection/recording any movements greater than 3 mm during the work.

   b. Optical transit survey, to determine benchmark elevations and corroborate manometer systems.

   c. Various crack monitoring devices which shall be set and remain in place for duration of the program to give continuous, visible indication of movement of structural members.

   d. A manometer system shall also be placed at various points on slope faces, where applicable (*slope is close to structure, slope is very steep, etc.*).

   e. Dial indicators to detect any movement of structural members, where applicable (*retaining walls, etc.*).

   f. Other monitoring equipment as deemed appropriate or necessary by the Engineer.

3. The Engineer shall monitor the grouting operation. The monitoring shall include review and approval of mix design, slump tests, strength tests, review of injection pressures and volumes, collect and review contractor's grout logs.

4. The contractor shall log each injection point during drilling and grouting, information contained in the logs shall include:

   a. Depth of penetration into firm undisturbed native soils as determined by the engineer, or as shown on the drawings.
   b. The volume of grout take at each stage. (0.3 to 1m stages)
   c. The depth of each stage. (0.3 to 1m stages)
   d. Maximum pressure obtained at each stage.
   e. Amount of uplift or slope movement obtained at each stage (if applicable).
   f. The final grout take at each probe location (sum of all the stages).

g. Remarks pertaining to that particular stage and the reason for termination of grouting.
h. Pumping rate in cubic feet per minute.

5. Contractor will distribute typed logs no less than once a week.

Methods

1. Grouting injection points shall be placed on a grid or spacing as determined by the Contractor (or Engineer) for the area to be grouted.

2. Top-down or bottom-up placement procedures shall be determined based on specific project needs. *Generally speaking, top-downs are used for releveling and bottom-ups for stabilization. Combinations of top-down, bottom-up placement procedures may be used to make placement more efficient and cost effective.*

3. The injection points shall be extended to depths that shall allow minimum penetration of *(fill in)* into undisturbed dense native formational material or as approved by owners engineer.

4. Alternate injection points shall serve as the primary sites of the pressure grouting program. Upon completion of the sequential grouting of primary injection points, secondary points shall then be grouted as determined by the engineer. The grout take of the secondary injections points shall be compared with the primary points. Grout take quantities should decrease in the secondary injection points. If the secondary grout takes are the same or greater than the primary grout takes, then additional injection points may be required by the soils engineer at additional cost to the owner unless otherwise stipulated in the bid documents.

5. Each of the injection points shall be cased with a casing that provides an adequate seal which shall eliminate movement of the casing and minimize grout return around the perimeter of the casing.

6. Grout shall be injected until one of the following criteria are met: *(Specify project specific requirement)*

    a. Undesired structure, slab, or ground movement occurs.

b. A predetermined volume of grout as has been approved by the Engineer has been injected in a given stage.

c. Pressure as measured at the probe header reaches:
   1. pressure determined by the engineer to be adequate.
   2. 4.2 MPa or greater when injection probe is less than 15 m from surface grades (including face of slopes).
   3. 5.5 MPa or greater when injection probe is more than 15 m from grade.
   4. Spike pressures of 10 MPa shall be considered refusal of initial take.

7. The rate of injection of the grout shall not exceed 60 liters/minute unless authorized by the engineer.

8. The contractor shall have the capability to restore the structure within a level condition upon completion of the compaction grouting. Level condition shall be defined "the Plans."

Basis of Payment/Units of Measurement

Payment should be made for the work based on the following unit prices:

1. Mobilization/Demobilization. The cost of assembling all plant, personnel, and equipment at the site prior to initiating the grouting program, and the cost of removing the same upon completion, will be paid in the contract lump-sum item for "Mobilization/Demobilization."

2. Grouting. Where the conditions are well defined, compaction grouting may be measured as a lump-sum price. Where conditions are not well defined, grouting should be measured for payment by the lineal meter for drilling, by the cubic meter for grouting, and by unit weights of constituent materials.

3. Routine construction quality control measures should be deemed included.

c. Jet Grouting

The following performance based specification is from the Central Artery Project in Boston, Massachusetts. It has been amended to be more general and useful for other projects.

Guide Specification for
Jet Grouting

1.01 General

A. This Section specifies requirements for creating soil-cement in situ by jet grouting to increase the compressive strength of subsurface soils over the depths and limits shown on the Drawings and for conducting a Quality Assurance-Quality Control program throughout the course of the Work to demonstrate that the installed soil-cement elements conform to requirements stated herein.

B. The jet grouting method shall be used to install soil-cement in areas indicated on the Drawings to form complete and continuous soil-cement elements. In using jet grouting, the achievement of full, intimate filling with soil-cement between adjacent soil-cement structures is a paramount requirement. Soil-cement shall be installed with the same make and model of; mixing machinery, cement grout mixing and pumping equipment, and the same materials and procedures implemented by the Contractor and accepted by the Engineer in the Pre-Construction (PPC) Program described in D below.

C. Jet grouting methods shall only be used at the locations shown on the Drawings and at other specific locations, authorized or directed by the Engineer.

D. Definitions

1. Testing Laboratory *(add agency)* responsible for forming, curing, preserving, transporting samples, performing laboratory testing, and reporting laboratory test results.

2. Surveyor - Registered Land Surveyor, responsible for measuring and reporting on all soil-cement location data.

3. Jet Grouting - The process of injecting cement grout to create an in-situ soil-cement. The cement grout is injected and mixed with the soil under high pressure through nozzles at the end of a monitor inserted in a borehole. The monitor is rotated at slow, smooth, constant speed which when combined with the rate of withdrawal achieves a continuous geometry and quality of soil-cement.

4. Soil-Cement - Homogeneous mixture of cement grout and in-situ soils.

5. Soil-Cement Element - A column of soil-cement formed by jet grouting which results in a homogeneous mixture of cement grout and the in-situ soils.

6. Soil-Cement Pre-Construction (PPC) Program - Field test program undertaken by the Contractor.

7. Spoil Return - All materials including, but not limited to, liquids, semi-solids and solids which are discharged above ground surface or mudline during, or as a result of jet grouting.

8. Obstructions - Man-made or man-placed objects or materials occurring at or below ground surface which unavoidably stops the progress of work for more than one (1) hour, despite the Contractor's diligent efforts. Obstructions include, but are not limited to, concrete, bricks, stone blocks, wood piles, metal, abandoned foundations, and utilities and other items. Known obstructions and areas of known obstructions are indicated in the Contract Documents either explicitly or by reference, although locations, configurations and the nature of known obstructions may vary from that stated. Unknown obstructions may not be indicated in the Contract Documents and are defined *(fill in appropriate reference)*.

   Naturally occurring materials such as: cobbles, boulders, dense, well-bonded or other competent in-situ soils will not be considered as obstructions. Cobbles, boulders, claystones, and sand and gravel layers may be encountered within the subsurface soils and will not be considered as either known or unknown obstructions.

E. Jet grouting shall be performed as established during the accepted PPC Program, but with adjustments during construction as necessary to fulfill requirements of paragraph 3.02.A.

F. A Quality Assurance-Quality Control Program shall be implemented during the course of the Work to confirm that the installed soil-cement achieves the required compressive strengths and plan area coverages over the depths and limits shown on the Drawings and alignment tolerances stated herein.

G. Related Work specified elsewhere:

(Complete as required)

1.02 Performance Requirements

The Contractor shall stabilize subsurface soils by jet grouting methods to provide continuous infilling, continuous masses or other necessary configurations of overlapping and interconnected soil-cement elements. Acceptance of soil-cement installation shall require documentation that the soil-cement has been installed: to the plan area coverages over the full depths and limits shown on the Drawings, to the alignment tolerances, and to the required compressive strengths and unit weights stated herein.

1.03 Qualifications

A. The jet grouting firm shall be experienced in jet grouting comparable to that described herein and have at least five years experience in jet grouting methods. Jet grouting experience shall include at least two projects of similar magnitude and complexity to that required for the program specified herein.

B. The jet grouting field superintendents shall each have at least three years experience in jet grouting techniques similar to that required for the Work specified herein; including at least two projects, one of which within the past five years of similar magnitude and complexity to that required for the program specified herein.

C. The Contractor's personnel responsible for survey layout, lines and grades shall be a Registered Land Surveyor or a Professional Civil Engineer.

1.04 Submittals

A. Submittals shall be stamped by a Professional Engineer, except for survey data which shall be stamped by a Professional Civil Engineer of a Registered Land Surveyor.

B. At least four weeks prior to commencement of any mobilization of jet grouting equipment for production mixing, submit the following to the Engineer:

1. Names and qualifications of the firms and personnel for the jet grouting firm and Surveyor, including project experience, resumes, and other documentation that demonstrate the qualifications of each field superintendent and rig operator for the jet grouting firms and each supervisor and field representative for the Surveyor.

2. A list of Owners, responsible engineers, and project descriptions from jet grouting projects completed within the past ten years. A list of owners, addresses, and telephone numbers shall be included for these projects representing the firm's comparable experience.

3. Additional information as follows:

    a. Equipment, procedures, and materials to be used for jet grouting. Equipment for overwater work including barges, support structures and equipment, anchors, boats, and work platforms. Include catalog cut sheets of: jet grouting equipment with dimensions and capacities of equipment and components; equipment for replacing hardened non-conforming soil-cement elements; cement grout mixing equipment; instrumentation used to measure and procedures to determine vertical alignment profiles; pumps; jet grouting monitoring systems; pipelines; spoil return casing; control equipment; and hoses and pumps; and barges and related spud, anchor and other stabilization devices.

    b. Spoil containment structures and methods to be used to prevent the migration of leakage of spoil return, disturbed in-situ soils or other spoil material beyond the immediate limits of soil-cement mixing operations. Include also details and methods to be used to collect and dispose of the spoil return and other spoil materials.

    c. Sequence and time schedule of all operations including plan location and sequence of jet grouting. The Contractor shall submit a jet grouting layout to achieve the required plan area coverages over the depths and limits shown on the Drawings. The pattern and sequence of jet grout shall be prepared to achieve actual overlap with each adjacent column, but maintaining center to center spacing of jet grout columns not greater than 75 percent of the smaller jet grout column diameter. Include as needed appropriate allowance for vertical alignment tolerances, or any other required inclined orientation. Plan locations of jet grouting shall be shown on Layout Plans of suitable scale to clearly show the details of the layout. Soil-cement elements shall be numbered and dimensioned to survey baseline established by the Contractor.

4. Mix design and mix procedures of soil-cement installed by jet grouting including: cement type, water-cement ratio by weight, and estimated minimum 56-day compressive strength. Also submit, proposed soil-cement element diameters, injection pressures and rates, withdrawal rates, and rotational speeds and calculations to demonstrate that full continuous columns of required diameter will be achieved in the site soils.

5. Methods and procedures to install soil-cement in the event unknown obstructions are encountered.

6. A Quality Assurance-Quality Control Program for jet grouting including, but not limited to the following:

    a. A detailed description of the Quality Assurance-Quality Control Program to be undertaken each day during jet grouting to confirm soil-cement conforms to: the plan area coverages over the depths and limits shown on the Drawings, horizontal and vertical alignment tolerances, and required compressive strengths and total unit weights specified herein.

    b. Details of the procedures to obtain soil-cement samples. Catalog cuts or shop fabrication drawings of the soil-cement sampling device and curing boxes.

    c. Measures to be implemented each day during jet grouting to continuously monitor, modify and control: water-cement ratios, cement-grout injection pressures and rates, rotational speeds, penetration and withdrawal rates, horizontal and vertical alignments and other related aspects of the jet grouting process.

    d. Example format of Daily Production Reports conforming to the requirements stated herein.

7. Proposed details and formats of all required tabular and graphical data presentations to be submitted to the Engineer during the course of the Work.

C. Within two business days after completing each soil-cement element submit to the Engineer:

1. Deviations of the center coordinate and diameter from the layout plan to the nearest 13 mm at the top of the element, and data to verify that the minimum column overlaps required in Article 3.02B are achieved full depth.

2. Elevations of the top and bottom of the soil-cement elements to the nearest 20 mm.

3. Vertical alignment profiles in accordance with the frequencies specified.

4. The Contractor shall compare the data to the requirements of Article 3.03 and report all deficiencies to the Engineers in the submittal.

D. Within one business day after the end of a work shift, submit Daily Production Reports for each work shift to the Engineer. Daily Production reports shall be filled out, checked for correctness, and signed by the jet grouting firm's field superintendent, the Contractor's field superintendent, and the Engineer at the end of every work shift. The reports shall contain but not be limited to, the following information:

1. Day, month, year, time of the beginning and end of the work shift; names of each superintendent in-charge of the Work for the jet grouting firm and the Contractor; a list of all workers' names associated with each jet grouting rig; and a summary of equipment used during the shift.

2. The location and limits of each completed soil-cement installed during the work shift and all soil-cement elements completed to-date on a plan of suitable scale to clearly detail the locations of the elements.

3. Time of beginning and completion of each soil-cement element installed during the work shift.

4. Water-cement ratios, cement type, brand and compound composition; a continuous strip chart record of cement grout injection pressures and rates, other pertinent cement grout mix data, mixing rotational speeds, penetration and withdrawal rates of the jet grouting equipment, and installation sequence for every soil-cement element including detail of jet nozzles.

5. Other pertinent observations including, but not limited to: spoil returns, cement grout escapes, ground settlement or heave, collapses of the soil-cement element, advancement rates of the jet grouting equipment, and any unusual behavior of any equipment during the jet grouting process and other noteworthy events. In the event of a Contractor claim, the Daily Production Reports shall be the primary documents to substantiate the reasons and basis for the claim.

6. Date, time, plan location, sample designation and elevation, and other details of soil-cement sampling.

7. Summary of any downtime or unproductive time, including start and end time, duration, and reason.

E. On the second business day of each week, the Contractor shall submit a revised layout plan showing deviations beyond contract allowable limits of elements installed, including those elements installed during the previous week.

F. Within five weeks after completion jet grouting operations, the Contractor shall submit an updated final Layout Plan of suitable scale and a compilation of all vertical alignment profiles.

G. Within two business days after performing work to remove or encapsulate unknown and known obstructions, submit a written disposition to the Engineer summarizing measures taken to remove or encapsulate obstructions including; date, time, nature, location and elevation of the obstruction, and plan location and elevation of all soil-cement or excavation and backfill performed to overcome the obstruction. A plan of suitable scale to clearly show the details and limits of the soil-cement or excavation and backfill shall be submitted to the Engineer.

H. Within seven business days of receiving the Contractor's submittals, the Engineer will review the submittals and notify the Contractor of deficiencies. The Contractor shall submit proposed actions and schedule which address the deficiencies within seven business days of receiving such notification.

1.05 Quality Control-Quality Assurance Program

A. The Contractor shall implement a Quality Assurance-Quality Control Program to verify the installed soil-cement elements conform to the requirements stated herein. The Quality Assurance-Quality Control Program shall be implemented as part of the Work, at no additional cost to the Department.

B. The Contractor shall obtain soil-cement samples, including wet grab and core samples, and provide them to the Engineer. The Engineer will form, preserve, cure, transport and test the soil-cement samples and report the test results. The Contractor shall cooperate with the Engineer and coordinate sampling activities an the Quality Assurance-Quality Control Program with the Testing Laboratory. The Contractor shall supply incidental items, access, inside storage space, curing boxes and electrical power to the curing boxes. The Engineer will supply molds for use in forming the samples.

C. The Quality Assurance-Quality Control Program shall include documenting all obstructions and the disposition of how each obstruction was overcome.

D. Wet Grab Soil-cement Samples:

1. A minimum of one in-situ sampling round, shall be performed at a frequency of once per day, at locations selected by the Engineer. The samples shall be obtained at the same element which shall consist of a non-cured soil-cement sample obtained at three depths selected by the Engineer. The Contractor shall obtain up to *(insert number)* additional wet grab sampling test suites at the direction of the Engineer at no additional cost to the Department.

2. Separate soil-cement samples shall be retrieved within 60 minutes of the withdrawal of the mixing equipment at a specific location. The device used to retrieve the wet grab soil-cement samples shall be capable of obtaining a discrete fluid sample of soil-cement at a pre-determined depth and shall be capable of accepting particles not thoroughly mixed that are up to 150 mm in any dimension. The sampler shall be lowered empty, air only, to the required depth in the soil-cement element and then opened. Once filled with the soil-cement the sampler

shall be closed to exclude entry or loss soil-cement and be expeditiously raised to ground surface.

3. Each retrieved soil-cement sample shall be of sufficient volume to produce a minimum of four full cylinders, 150 mm diameter by 300 mm height. Separate and retain all soil-cement retrieved from each depth, and immediately provide each to the Engineer. The Engineer will then cut all retrieved particles of soil larger than 25 mm into smaller pieces that pass a 25 mm sieve and then form immediately the four cylinders of material passing through a 25 mm sieve.

4. Soil-cement samples shall be protected from freezing and extreme weather conditions which could have deleterious effect, at all times in accordance with AASHTO T 23.

5. Soil-cement cylinders from each sampling depth, selected by the Engineer, will be tested to determine the 7 day and 28 day unconfined compressive strength in accordance with AASHTO T 208. The Contractor may obtain additional samples and perform additional testing for his own information, at no additional cost to the Department.

6. If the Contractor cannot obtain all of the required wet grab samples of the soil-cement, in the designated soil-cement element, the Contractor shall obtain a full suite of wet grab samples from the next soil-cement installed by that rig. Continue taking wet grab samples in subsequent soil-cement elements until a full suite is obtained.

E. Core samples:

1. As directed by the Engineer, core samples shall be taken for the purpose of obtaining and testing in-situ samples to evaluate compressive strength, unit weight, and composition of the soil-cement. Coring of soil-cement shall be performed with a triple tube core barrel with a side discharge in accordance with AASHTO T 225 and the requirements stated herein.

2. Continuous core samples shall be obtained at up to *(fill in)* locations, selected by the Engineer, over the entire depth of the soil-cement. The samples shall be obtained using a PQ-size triple core barrel with a side discharge.

3. Immediately after retrieving the soil-cement core samples from a specific boring, wrap, preserved and submit to the Engineer seven core samples per boring, selected by the Engineer for subsequent evaluation and testing. Grout the borehole after coring.

4. Remaining core samples shall be boxed, stored, preserved and delivered to the Engineer. Core samples shall be protected from freezing and extreme weather conditions at all times.

5. If recovered core samples from any boring provide less than 90% recovery, or less than 50% RQD, or fewer than two intact cores of length more than eight inches, in each core run, the Engineer may direct the Contractor to drill up to 2 additional borings and recover additional core samples for testing. If the samples from either of these additional borings do not provide required recovery, this process shall be repeated until coring provides required samples. These additional borings and core sampling shall be done at no additional cost to the Department.

6. The Contractor may retrieve additional samples and perform additional testing for his own information, at no additional cost to the Department.

F. Vertical Alignment Profiles

1. The Contractor shall obtain vertical alignment profiles over the length of one soil-cement element per day, along two perpendicular axes as directed by the Engineer.

2. The Contractor shall advise the Engineer within one hour after measuring the vertical alignment, of any non-compliance with tolerance requirements. If any soil-cement element exceeds allowable vertical element tolerances,, the vertical alignment of the next soil-cement element installed by the rig shall be measured at no additional cost to the Department.

3. The Contractor shall obtain up to 50 additional vertical or inclined alignment profiles at the direction of the Engineer at no additional cost to the Department.

1.06 Job Conditions

A. Subsurface strata may contain *(fill in description of profile)*. Known obstructions and areas of known obstructions are shown in the Contract Documents.

B. The Contractor shall take all precautions necessary to prevent movements and damage to any existing structure, roadway and utility, and also prevent settlement or heaving of ground that could occur due to jet grouting operations in the vicinity of existing structures, roadways, railroads, bridges, seawalls or utilities.

C. Other general site conditions include, but are not limited to the following:

(Describe as necessary)

2.01 Cement Grout

A. Cement shall conform to AASHTO M85, Type II, and shall be as specified in the submitted cement grout mix design accepted by the Engineer. Slag cement or flyash shall not be allowed without the acceptance of the Engineer and submission by the Contractor of test data which confirm no deleterious impact to the soil-cement.

1. 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. 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 staked no more than ten bags high to avoid compaction. Cement containing lumps or foreign matter of a nature and in amounts that may deleterious to the grouting operations shall not be used.

3. All cement shall be homogeneous in composition and properties, and shall be manufactured using the same methods at one plant by one supplier.

4. Tricalcium aluminate content shall not exceed 8 percent.

B. Water used in jet grouting, mixing grout and other applications shall be potable, clean, and free from sewage, oil, acid, alkali, salts, organic materials and other contamination.

C. Grout admixtures shall not be allowed without the written approval of the Engineer.

D. Cement grout shall be a stable homogenous mixture of cement and water. The ratios of the components shall be proposed by the Contractor, confirmed during the SCIC Program, and reviewed by the Engineer. Cement grout composition shall not change throughout the soil-cement mixing unless requested in writing by the Contractor and accepted in writing by the Engineer.

2.02 Soil-Cement by Jet Grouting

A. Soil-cement shall be a stable homogenous mixture of cement grout and in-situ soils. The properties listed shall be verified throughout the course of the Work in accordance with the Contractor's Quality Assurance-Quality Control Program.

B. The ratios of various soil-cement components shall be proposed by the Contractor or as accepted during the PPC Program subject to review by the Engineer. The Contractor shall adjust the mix design throughout the course of the Work in order to achieve the required compressive strengths and total unit weights. The Contractor shall submit changes in the mix design or cement factor and obtain the Engineer's acceptance prior to implementing these changes. The Contractor shall also adjust the mix design when directed by the Engineer.

C. Soil-cement obtained from wet grab samples shall conform to the following minimum compressive strength requirements. Unconfined compressive strength testing shall be performed in accordance with ASTM D2166.

1. Soil-cement shall achieve a 28 day *(or 56-day)* unconfined compressive strength of f'c equal to *(fill in)*.

2. The average unconfined compressive strength within each soil cement element shall be *(fill in)*.

D. The total unit weight of soil-cement samples shall be measured and shall be at least *(fill in)* or as determined and accepted by the Owner. For each test suite the average total unit weight of all soil-cement samples within the round will be calculated. If the average total weight from any two consecutive test rounds is less than *(fill in)* the Contractor shall adjust its mix as necessary to achieve the required unit weight.

E. Conformance with soil-cement uniformity criteria will be determined by the Engineer by evaluation of core samples. The soil-cement shall contain soil fragments with a maximum dimension not to exceed 1/4 of the diameter of the auger or 300 mm whichever is smaller. In addition, seventy percent of the depth cored shall have a minimum core sample unconfined compressive strength of *(fill in)*.

Note that strength evaluations for quality control based on core samples of a given age (28 or 56 days) may be specified in lieu of strength based on wet grab sampling. A maximum unconfined compressive strength limit may be specified if the soil mix volumes are subsequently excavated.

2.03 General Equipment Requirements

A. All equipment shall be maintained to ensure continuous and efficient production during jet grouting.

B. The jet grouting equipment shall be provided with specialty drilling bits capable of advancing through the site subsurface conditions including, but not limited to, concrete, brick, granite stones and blocks, wood, timber piles, cobbles and boulders.

C. The cement grout batching plant shall include all storage cribs, weather protection, sheds, scales pumps, mixers, valves, gauges and regulating devices required to continuously measure and mix cement grout. The cement grout mixer shall be a high-speed colloidal type capable of operating at up to 1,500 RPM.

D. The jetting system shall utilize up to eight radially oriented jet nozzles to separately inject cement grout, water, and/or (if used combined) air/water or air/grout fluid. Air shall not be injected by itself into the in-situ soils. The grout jet nozzles shall be easily and quickly adaptable with orifices between 1.0 mm and 4.0 mm in diameter. The jet grouting equipment shall be capable of providing at least 48 MPa at each nozzle, but if necessary for the Contractor's particular equipment, the higher pressures shall be used. The equipment shall provide for continuous positive return flow using sacrificial casing or retractable pipe casing during jet grout operations.

E. The system shall be capable of variable rotation and withdrawal rates within the ranges as necessary to complete the work and produce the required continuous soil-cement elements.

F. Equipment instrumentation shall be provided to allow continuous monitoring and recording of data throughout the jet grouting operations. As a minimum, the following shall be provided versus depth:

1. Pressure gauges at the drilling rig, and recording devices to record cement grout injection and other fluids pressures during grouting.

2. Flowmeter to monitor and record the rate and total volume of cement grout injection and other fluids through the grouting monitor at every soil-cement element.

3. A means of measuring and recording the rate of return flow out of the drilled hole and total volume of return flow from the grout hole during jet grouting.

4. A means of monitoring and recording the rate of rotation and rate of withdrawal of the monitor.

G. The contractor shall provide equipment capable of replacing hardened, non-conforming elements.

3.01 General

A. The Contractor shall furnish sufficient equipment, materials, and labor necessary to conduct the required jet grouting operations to complete the Work in accordance with project schedule and milestones.

B. The Contractor shall conduct all survey layout, and utility clearance for the jet grouting operations and coordination with all local, state, and federal agencies having jurisdiction.

C. Within 14 days after receiving the Engineer's notification to proceed with jet grouting, the Contractor shall mobilize to the site the required equipment, labor, and materials required and labor and begin substantive jet grouting activities.

D. The Contractor shall coordinate jet grouting operations with other aspects of the Work.

E. The Contractor shall install jet grouting in a manner so as to not create obstructions or other hindrances to subsequent aspects of the Work.

3.02 Installation of Soil-Cement

A. Install soil-cement by jet grouting using the same make and model of; mixing machinery, cement grout mixing and pumping equipment, and the same materials and procedures implemented by the Contractor and accepted in the PPC Program.

B. Soil-cement shall be installed in accordance with the patterns developed by the Contractor and accepted by the Engineer, to achieve the required plan area coverages over the depths and limits shown on the Drawings, compressive strengths, and unit weights stated herein. Center-to-center spacing of jet grout elements shall at any elevation not be greater than 75 percent of the jet grout column diameter and shall include allowance for tolerable vertical alignment deviation.

C. Soil-cement shall penetrate to approximately *(fill in project specific requirement)*. Final grouting of the soil-cement elements shall not commence until the Engineer has approved the bottom depth of the soil-cement element. Non-conforming elements shall be replaced at no additional costs to the Department. Where developed, soil-cement strength precludes replacement using jet grouting equipment, the Contractor shall provide alternate equipment to replace non-conforming elements.

D. *In the event geotechnical, or other types of, instrumentation indicates movements related in any way to the jet grouting operations, the Contractor shall meet with the Engineer and implement a proposed action plan to restrain and control movements within allowable ranges. The Contractor shall stop jet grouting operations if necessary in order to limit movements or implement the proposed action plan.*

E. After final grouting of the soil-cement elements, the Contractor shall obtain samples of in-situ soil-cement in accordance with the locations and frequencies required in the Quality Assurance-Quality Control Program.

F. Any soil-cement element which exhibits partial or total instability, shall be backfilled with weak cement grout and be remixed full-depth, at no additional cost to the Department.

G. Once jet grouting is started at any location, the jet grouting operation shall continue until the soil-cement element is completed.

H. Soil-cement elements shall not be installed within 600 mm as measured between outside edges of soil-cement elements that are less than 48 hours old. The 48 hour delay may be shortened if the Contractor demonstrates to the satisfaction of the Engineer that the installation of any adjacent placements would not have a deleterious effect on any previously installed soil-cement elements or the ground.

3.03 Horizontal and Vertical Alignment Tolerances

A. The maximum horizontal deviation of the as-installed center of any soil-cement element at the ground surface or mudline installation level shall not exceed 75 mm from the layout center coordinate, shown on the accepted Contractor's submittal.

B. The vertical alignment of soil-cement elements shall not deviate in any direction more than 2 percent from vertical.

C. At the direction of the Engineer, any soil-cement element which exceeds the allowable horizontal or vertical alignment tolerances shall be re-mixed within two days or initial placement, or supplemented with one or more adjacent or overlapping elements, at no additional cost to the Department.

3.04 Obstructions

A. Subsurface strata, may contain rubble, concrete, reinforced concrete slabs, metal,, bricks, granite, stone and blocks, wood piles, seawalls, railroad track beds, abandoned foundations, utilities and other materials that can obstruct jet grouting operations.

Naturally occurring materials such as: cobbles, boulders, dense, well-bonded or other competent in-situ soils will not be considered as obstructions. Boulders, claystones, and sand and gravel layers may be encountered within the subsurface soils but will not be considered as either known or unknown obstructions.

B. Where unknown obstructions are encountered during the jet grouting, the Contractor shall remove the obstruction or install additional soil-cement to encapsulate the obstruction, at the direction of the Engineer. Each situation shall be resolved and paid on a case by case basis. If such conditions are encountered, the Contractor shall notify the Engineer in writing, and provide all pertinent information relating to the nature, depth, plan location coordinates, expected extent of the obstruction, and proposed procedures to overcome the obstruction.

C. If difficult drilling is encountered due to the presence of naturally occurring cobbles, boulders, or dense well-bonded in-situ soils, or other characteristics of the in-situ soils, the Contractor may elect to remove the object or submit an alternate jet grouting layout pattern to avoid or encapsulate the object, subject to the acceptance of the Engineer and at no additional cost to the Department. Such naturally occurring conditions shall not be the basis for additional measurement or compensation.

3.05 Containment, Collection, and Disposal of Spoil Return

A. At all times during jet grouting operations, the site shall be maintained cleared of all debris and water. Spoil return shall be piped or channeled to holding ponds, tanks or other collection structures. The Contractor shall regularly dispose of all waste materials in accordance with the requirements of the Department of Environmental Protection and all other agencies having jurisdiction.

B. All soil-cement collection, containment and disposal methods shall be shown on shop drawings in the Contractor's submittals to the Engineer prior to the start of jet grout operations. The Contractor shall be responsible for and incorporate all sedimentation and turbidity control measures required by applicable federal, state and local regulations.

C. The Contractor shall take all necessary precautions and implement measures to prevent any spoil return, other spoil material or stockpiled materials from entering storm drain structures, drainage courses and other utility lines or from leaving the site via surface runoff. The Contractor shall prevent the migration of spoil return, spoil material or stockpiled materials into any surface water body, beyond the immediate limits of soil-cement mixing operations.

4.01 Method of Measurement (Note: this applies to relatively straight forward cases where the volume of soil to be grouted is well defined. Other alternatives should be considered in more complex situation.)

A. Installation of soil-cement by jet grouting will be measured per cubic meter to the nearest cubic meter, within only the "neat" plan area of the proposed soil-cement shown on the

Drawings or approved by the Engineer. The volume shall be determined by multiplying the "neat" area within this zone times the actual depth of the soil-cement.

Additional quantities of soil-cement installed to overcome unknown obstructions shall be incorporated into the total measured quantity of soil-cement, as accepted by the Engineer.

Quantities of soil-cement installed by the Contractor during remixing to achieve the performance requirements, or that are outside the limits of soil-cement mixing shown on the Drawings without the accepted of the Engineer will not be measured for payment.

B. No separate measurement will be made for; the Contractor's Quality Assurance-Quality Control Program, the supplemental full depth test suites of wet grab samples, the supplemental vertical alignment profiles, all of which shall be considered part of the Work of jet grouting except for core borings.

C. Removal of obstructions will be measured for payment under Section *(xx)*.

D. Core borings will be measured for payment under Section *(xx)*.

E. Mobilization and demobilization will be paid under Section *(xx)*.

4.02 Basis of Payment

A. Installation of soil-cement by jet grouting will be paid as a unit price per cubic meter and will include full compensation for furnishing all equipment, materials and labor required to install soil-cement in accordance with the plan area coverages over the depths and limits shown on the Drawings and requirements herein, and to implement the Contractor's Quality Assurance-Quality Control Program, and make the supplemental wet grab samples and vertical alignment profiles.

4.03 Payment Items

120.904	Installation of Jet Grout	Cubic Meter

d. Soil Fracture Grouting

Since very few projects have been completed in North America so far it is not possible to provide a very detailed guide specification at this time. The following highlights areas that need to be addressed in a specification.

Guide Specification
Soil Fracture Grouting

Qualifications:

- The contractor must have performed at least two soil fracture projects successfully completed within the past five years.

- Employ key personnel that should include a project manager and project superintendent who have provided full-time, on-site supervision of at least one soil fracture project.

- Have experience performing first order, real-time remote movement monitoring during grouting.

Execution

Drilling

Suitable drill rigs should be furnished for installation of the grout pipes with a drilling system to penetrate the formation without causing caving or collapse. The drilling procedures used for grout pipe installation should be suitable to penetrate all strata while ensuring drill hole stability

Installation of Grout Pipes

Grout pipes may be installed horizontally, inclined, or vertically to obtain the specified minimum grout coverage. After being placed in a borehole, the sleeve-port grout pipes should be encased in a continuous cement bentonite grout sheath. Installation of pipes should allow for sufficient redundancy to compensate for abortive grout ports and pipes. All grout pipes should be accurately surveyed for alignment using a suitable computer-based borehole-survey tool able to verify the required grout pipe layout: Additional grout pipes should be installed as necessary to provide grout sleeve coverage to critical areas missed by drilling misalignment.

Movement Monitoring

The Contractor should design, supply, install, and operate a structural movement monitoring system consistent with the flexibility of the structure and the Engineer's criteria for settlement and angular distortion. Monitoring of movement must be at a rate faster than the predicted rate of maximum settlement.

For large sensitive structures, a monitoring system capable of reading and processing movements in real time with sufficient accuracy to identify angular distortions of 1 in 2,500 is required. Such a system would comprise of a series of electro-levels or hydraulic overflow gages.

Precondition Grouting

Precondition grouting is the initial grout injection to reinforce the soils before the need for reactive compensation grouting in response to settlement. The precondition phase shall be repeated until slight heave is measured at the structure and to the extent that further injection results in rapid, controlled heave.

Compensation Grouting

Compensation grouting is the reactive process by which soil fracture is used to both arrest and counteract observed settlements. Grout should be injected to compensate for settlement as recorded by the movement monitoring system. Repeated, discrete injections must be capable of lifting the structure at a rate and magnitude greater than that of the predicted maximum settlement. Grout injection tubes should be flushed clean to permit post-construction grout injection for control of long-term settlements as required.

Trial Grouting Program

A trial grouting program should be developed and implemented prior to beginning any production grouting. Details of the trial grouting program, such as grout tube spacings, installation, materials, procedures, and equipment should be identical to that intended to be used on the production work. The Contractor will install at least two ground settlement monitoring points at suitable locations in the test area. The two points should be surveyed and tied into the project control monuments. The grouting test should be suitably designed to accomplish the following objectives:

- Demonstrate the ability to lift two points 25 mm simultaneously and to maintain angular distortion to within 1/1,500.

- Demonstrate the ability to lift the two monitoring points at a rate equal to the maximum rate of settlement calculated for the tunnel boring machine.

Submittals

Sixty days prior to the start of the drilling work, the grouting contractor should submit a detailed grouting plan for the work, specifying the locations and grout-pipe installation procedures. The work plan should show the basis of establishing grout target volumes for both preconditioning and compensation grout phases.

Basis of Payment/Units of Measurement

Payment should be made for the work based on the following unit prices:

1. Mobilization/Demobilization. The cost of assembling all plant, personnel, and equipment at the site prior to initiating the grouting program, and the cost of removing the same upon completion, will be paid in the contract lump-sum item for "Mobilization/Demobilization."

2. Grouting. Installation and testing of the monitors should be measured as a lump sum price. Drilling and installation of grout pipes should be measured per lineal meter. All precondition grouting should be measured as a lump sum price or on a time and material basis. Compensation grouting shall be measured on a time and material basis.

e. Lime Injection

Guide Specification
Lime Injection Grouting

Scope of Work

In connection with the lime injection program as shown on the contract plans, the contractor should provide all labor, materials and equipment to accomplish the following items of work:

- Mobilizing for the injection.

- Performing a test program to confirm effectiveness of the injection program.

- Performing specific tests, i.e. moisture content and pocket penetration, to confirm the effectiveness of injection.

- Site clean-up and disposal of waste slurry resulting from the injection.

Qualifications

The work of this section is specialized and should be contracted to an expert firm with an established record of lime injection applications.

An experienced, full-time supervisor who has been in responsible charge of supervising lime injection operations for at least five years should be assigned. This supervisor should be present at the work site at all times during injection operations.

Submittals

Working drawings and method descriptions should be given that provide the following information:

- Plant, equipment and material descriptions.

- Arrangement of grout mixing and injection equipment, injection point locations, and other necessary details.

- Methods for drilling and supporting injection holes.

- Sequence of lime/fly ash injection.

- Slurry mix design, sources of mix materials (i.e. lime/fly ash), and material data demonstrating compliance with the grout materials specified in the contractor's recommendations. Lime should be ASTM C977 and fly ash should be Type 'C.'

Injection Records

Lime injection records should be submitted daily during the execution of the test and production lime injection programs. The daily reports should contain the following information:

- Injection hole geometry.
- Time and date of beginning and completion of each injected area.
- Injection mix data, including ratio of injected materials, and specific gravity.
- Pressures used during each injection.
- Injection rates and takes for each area.
- Test results obtained from soil treated in the test program and during actual injection. Other pertinent observations such as injection refusal, water displacement, or other unusual behavior.

Injections Procedure

Phase I

Prior to the start of injection stabilization, the building pads or injection areas should be brought to grade and staked out to accurately mark the areas to be injected. Depending on soil properties and in-situ moisture, provisions should be made for 50 mm to 200 mm of swelling that may occur as a result of the injection process.

After the pads or areas have been brought to grade, the following procedures are usually followed.

- Injection pressures should be controlled within a range of 350 to 1,400k Pa.
- The slurry should be mixed at a rate of 0.5 kg of lime per liter of water to produce a specific gravity of approximately 1.150 to 1.180 at 20° C. If quicklime is slaked, the specific gravity of the elevated temperature slurry must be adjusted to compensate for the decrease in density of water at slurry temperatures of 80 to 90°C using an appropriate

conversion table. The injection contractor should provide a hydrometer, Baroid Scale or other suitable method to accurately verify slurry mixes.

- Injections should be made in two passes, each spaced 1.5 m on center in each direction to create a 0.5 meter diagonal offset pattern. Injections should be extended a minimum of five feet outside the treatment area.

- Lime slurry should be injected to a depth of 2.5 to 3.0 m or to impenetrable material, whichever occurs first. Impenetrable material is the maximum depth to which two injection rods can be mechanically pushed into the soil using an injection machine having a minimum gross weight of 4,500 kg.

Injections should be made in 0.3 to 0.5 m intervals down to the total depth with a minimum of 4 stops or intervals. The lower portion of the injection pipes should contain a hole pattern that will uniformly disperse the slurry in a 360 degree radial pattern. Injections should be continued at each interval until refusal (i.e. the maximum amount of slurry has been injected and the slurry is running freely at the surface, either out of previous injection holes or from areas where the surface soils have fractured), or until a maximum of 35 kg of hydrated lime per square meter of treated area has been installed, whichever comes first.

The quantity of lime injected should be closely monitored on a daily basis in an effort to achieve a uniform distribution throughout the treated area. For estimating purposes, the total quantity (kg) of lime to be injected should be determined as follows:

$$(A * Q) / 450 = \text{kgs of lime needed},$$

where A is the surface area injected and Q is the estimated quantity of lime in kgs per square meter.

If the total amount of lime injected is less than the specified amount, the injection contractor shall, at no additional expense to the customer, reinject the deficient amount of lime uniformly over the entire treated area or around the perimeter of the treated area, as determined by the engineer.

- After completion of the Phase I lime injection and prior to the start of Phase II, the surficial lime should be mixed into the top 100 mm to 150 mm of soil and lightly compacted to seal the surface and to form a working platform.

Phase II

- Following completion of the base-bid lime injection work specified in Phase I, the entire area should be injected with water and surfactant at locations offset from lime injection points and to the same total depth as specified for the lime injection.

- Injections should be made in approximately 300 mm to 460 mm intervals from the surface down to the specified depth, injecting to refusal at each interval. (NOTE: While conditions will vary from one site to another, as a general rule approximately 45 liters of water per cubic meter can be injected on a single pass.) The total number of water injection passes required will be determined by the soils engineer or the customer. Refusal criteria are as for lime injection.

- A minimum of forty eight hours should be allowed between all injection phases.

Acceptance Criteria [45]

Acceptance criteria for lime injection vary depending on the performance criteria for the project. Any one of the following criteria can be used:

- A moisture content of at least the plastic limit plus 2 to 3 moisture points.

- An average swell of 1.0 percent for all swell samples taken with no more than 20 percent of samples greater than 2.0 percent.

- A hand penetrometer value of 290 kN/m^2 or less.

- No acceptance criteria specified for the project.

- Terminate the lime injection work after four injections.

- One or more of the above.

Basis of Payment/Units of Measurement

Payment should be made for the work based on the following unit prices:

1. Mobilization/Demobilization. The cost of assembling all plant, personnel, and equipment at the site prior to initiating the grouting program, and the cost of removing the same upon completion, will be paid in the contract lump-sum item for "Mobilization/Demobilization."

2. Grouting. Lime injection grouting should be measured as a cost per surface square meter.

6.3 ROCK FISSURE GROUTING SPECIFICATIONS

a. Specification Elements

Regardless of the amount of geologic knowledge gained during the design exploration studies, it is almost certain that more will be learned during construction and as the grout holes are drilled. It is almost equally certain that some adjustments to the grouting program will be appropriate as a result of what is learned. Therefore, it is essential that the specifications provide the flexibility that is needed in order to accommodate those adjustments. Circumstances commonly arise in which it is appropriate to have some of the work done on a time and materials basis (i.e. there probably will be “changed conditions”). If this is not done, the working relationship between the owner’s representative and the contractor may deteriorate. Ideally, the specifications should provide prospective bidders with an opportunity to present alternative approaches, materials, and/or equipment for performing the work.

Most rock grouting projects have been for dam curtain and tunnel grouting applications. In these cases, the specifying agencies have considerable experience as to their needs and have utilized method specifications. For an unusual rock grouting project, a performance specification may be preferable.

Specifications for rock foundation grouting should always include the considerations and provisions discussed briefly below, and are frequently on the order of 30 pages or more. The following highlights areas that need to be addressed.

Drilling Equipment

Rotary or rotary percussive methods with water flush may be permitted generally. Few circumstances, other than access problems, justify use of diamond drilling of grout holes, which is much slower and more expensive than rotary percussion drilling. The drilling equipment must be designed or modified to enable the drilling to be done entirely with water circulation. (Use of foam may be allowed if water alone cannot keep the hole clear of cuttings.) The contractor should be required to employ such special drill rods or other special equipment needed to meet specification tolerances. Due to the potential for erosion of the borehole walls and resultant difficulty in seating packers, use of side-discharge bits should not be allowed except in fresh, hard rock.

Drilling below any point at which there is an observable loss of circulation of drilling fluid should be specifically prohibited unless the MPSP system is to be used. If not, the subsurface openings that caused that loss quickly become ungroutable as they become clogged with rock cuttings.

Grouting Equipment and Procedures

Use of "colloidal" mixers should be specified. Use of progressive cavity pumps should not be an absolute requirement. Automatic recording equipment should be used. The specifications should require that the contractor take all necessary and appropriate measures to ensure the accurate proportioning of grout materials so that a consistent product - as determined by frequent field tests of viscosity and specific gravity - is achieved (and demonstrated).

b. Construction Control

General Considerations

Regardless of the number of exploratory borings or other preconstruction investigations, information on the size and continuity of groutable, natural openings in the rock mass will be relatively meager at the start of grouting operations. The presence of groutable paths can be ascertained before grouting and can be verified during grouting, but the sizes, shapes, and interconnections of these paths will be largely conjectural. The art of grouting consists mainly of being able to satisfactorily treat these relatively unknown subsurface conditions without direct observations. One of the benefits of most grouting programs is exploration. A carefully monitored and analyzed drilling and grouting program can provide significant information about a foundation. However, data must be correlated and analyzed to be of any benefit.

Grouting procedures depend on the project, policy, objective, geology, contractor, field personnel, and individual judgment and preference. Procedures subject to variations depending on project conditions include drilling, washing, pressure testing, selection and adjustment of mixes, changing grouting pressures, flushing the holes and washing the pump system during grouting, determining the need for additional grout holes, treatment of surface leaks, and maintaining up-to-date records of drilling, grouting, and monitoring.

Drilling Operations

Since drilling is a vital and costly part of the foundation grouting program, a daily drilling record of all pertinent data should be kept by the inspector during the operation.

Grouting Operations

- Washing holes. Washing of grout holes immediately prior to the injection of grout is necessary. The purpose of the washing is to remove all drill cuttings and mud from the grout holes and to flush cuttings, sand, clay, and silt from the fractures in the rock. These materials must be removed to the maximum extent possible in order that the grout can be injected and that windows may not later be eroded in the grout curtain in rock fractures at the time of the grout injection.

- Pressure Testing. Pressure testing is performed as part of the pressure- washing operation. Its purposes are to obtain an indication of the permeability of the foundation, to determine the location of permeable zones, to verify seating of the packer for pressure washing, and to evaluate the effectiveness of the pressure washing. Adjacent holes are uncapped during the test to allow venting of the water. Each grout stage must be pressure tested. The test should be initiated prior to pressure washing by injecting only water into the hole for a minimum of 10 minutes under a steady pressure. The rate of inflow should be measured each minute. After 10 minutes, and if the test indicates that passages in the formation are being opened by the water, pressure washing should be initiated. For better definition of conditions, a multipressure test should be used.[3]

Rising injection pressures during grouting should be controlled by varying rates of injection and grout mix formulations so that they slowly rise in increments until the desired refusal criteria are reached. If the injection rate suddenly increases with a drop in pressure when grouting at the maximum safe pressure, lifting may be suspected and appropriate precautions taken.

A maximum pumping rate should be established to restrain grout travel within reasonable limits and to have better control of the job. Usually, 0.085 m^3 per minute is considered a reasonable maximum pumping rate for most foundation grouting.

Split spacing is the correct way of sequencing the work. The decision on upstage, downstage, or MPSP is site specific.

Geologic profiles should be kept up to date with drilling, testing, and grouting data, and records should be made of monitoring data to evaluate the ongoing grouting program. This information should also be included in the as-built report for future reference.

Evaluation of grouting effectiveness must be constant and continuous throughout the program, and should be a joint effort between engineering and construction personnel. If problems develop, reaction should be expeditious. Flexibility must be maintained for making changes and improvements as the program progresses. Design changes of other project features are sometimes made based on knowledge of foundation conditions gained during grouting.

Completion of Grouting

Grouting should be continued to absolute refusal (zero flow rate condition at maximum allowable injection pressure) although this is not usually done. Typically, the project designer will determine the maximum injection pressure per meter of depth and the refusal flow rate, depending on the geology and the program objectives.

6.4 VOID FILLING SPECIFICATIONS

It is often difficult to obtain sufficient exploratory information to accurately define the extent of underground cavities. Therefore, the drilling and grouting operation should be considered as a continuation of the design subsurface investigation. A method specification can be utilized, with quantity estimates for drilling and grouting for bidding purposes and provision made for quantity changes that will typically occur. A method specification can be used for scour repair utilizing fabric tubes. In slabjacking projects, the tolerance in the final slab elevation is critical, and performance specifications are preferred over method specifications, assuming the outside variables such as traffic can be controlled. Void filling by grouting alone will follow the major points for fissure grouting (above).

a. Specifications

Guide Specification
Concrete Filled Forms For Scour Repair
(From PENN DOT)

Description

This item should govern for the construction of concrete-filled containers for scour repair in accordance with these specifications and with the lines, grades, design, and dimensions shown on the plans or established by the engineer.

Synthetic textile forms are employed as forms for concrete units. The units are pumped in place and connecting dowels are used to ensure interlocking between the tubes or bags.

Forms

Containers (tubes or bags) for concrete placement should consist of a woven geotextile from stabilized yarns.

Each container should be designed to remedy each particular scour zone when pumped with concrete or in such a way that when a group is placed together, the scoured area is protected. These containers should be constructed with a minimum of one self-sealing valve to facilitate concrete pumping. If there is uncertainty in the scour void dimensions, tubes and bags should be field sewn to ensure that the height of the inflated concrete containers will not be more than one-half of the width.

The geotextile should meet the requirements listed in table 7.

Table 7. Required minimum property values for geotextiles

Physical Property	Test Method	Unit	Values
Composition			Polypropylene
Weight (double-layer)	ASTM D3776	g/m	7.5
Thickness	ASTM D1777	mm	23
Mill Width		mm	80/165
Mechanical Property			
Grab Tensile Strength	ASTM D4632	N	320 Warp - 300 Fill
Grab Tensile Elongation	ASTM D4632	percent	18 Warp - 22 Fill
Burst Strength	ASTM D3786	Pa	625
Trapezoidal Tear Strength	ASTM D4522	N	130 Warp - 130 Fill
Puncture Strength	ASTM D4833	N	80
Hydraulic Property			
Water Flow Rate	ASTM D4491	$L/min/m^2$	105
Coefficient of Permeability	ASTM D4491	mm/sec	0.9
Permittivity (k/l)	ASTM D4491	1/sec	1.5
Porosity	ASTM D737-75	$m^3/min/m^2$	300

Reinforced Dowel Rods (if required)

Reinforcing dowels will be constructed of stainless steel or an approved equal. The type and strength of the rods should be submitted to the engineer for prior approval. Rods should be embedded at least 0.3 m into the lower bag or tube and protrude 0.3 m into the upper bag or tube at each location. For tubes, these dowels should be spaced one meter apart on center.

Guide Specification
Subsealing Concrete Pavement Using Particulate Grouts

Description

Under this item, the contractor should prepare and place a grout in amounts sufficient to fill all voids beneath the pavement slabs at locations indicated on the plans and as directed by the engineer. Voids should be filled without affecting the final grade of the pavement.

Materials

The subsealing slurry should be proportioned as follows:

One part by volume Type I or Type II portland cement
Three parts by volume pozzolan
Water to achieve required fluidity.
Bentonite or other additives may be permitted to achieve appropriate fluid and set characteristics.

The mixture should have the following properties:

- Minimum compressive strength of 2,000 kPa at 7 days

- Efflux time of 16 to 26 seconds. The subsealing slurry's fluidity should be measured by a method similar to the Army Corps of Engineers flow cone method CRD-C79.

The contractor should submit to the engineer a written proposal for approval, which should include a physical and chemical analysis for the pozzolan and independent laboratory test results for the pozzolan subsealing slurry showing 1-day and 7-day strengths, flow cone times, shrinkage and expansion observed, and time of initial set. The engineer should have 10 working days to render a decision on acceptability.

Equipment

The contractor should furnish all equipment necessary for storing, transporting, accurately batching, mixing, testing, and pumping the subsealing slurry to fill the voids beneath the pavement. All equipment should be approved by the engineer before its use.

For mixing the pozzolan slurry a high speed colloidal mixer is required. The mixer should operate at speeds between 800 and 2000 rpm and should consist of a rotor operating in close proximity to a stator, creating a high shearing action and subsequent pressure release to make a homogenous mixture.

Construction Details

The contractor should schedule this operation such that successive drilling and grouting operations will progress within a time period that will ensure the proper maintenance of traffic.

The time of efflux of the subsealing grout should range from 16 to 26 seconds. A more fluid mix having a flow cone time of 9 to 15 seconds may be used during the initial injection at each hole. These measurements should be made by the contractor, under the supervision of the engineer, at least two times per day or as directed by the engineer. The contractor should supply the test equipment to measure the efflux times.

The pressure distributor should be approved by the engineer and should be capable of pumping the subsealing slurry at sufficient pressure over the required distances and through the drilled holes into the voids beneath the slabs. Holes may be washed with water to create a small cavity to allow the initial spread of the grout. Pumping should continue until the mixture emerges along the edge of the pavement or breaks through the shoulder. To prevent distortion of the slab, pumping should be immediately discontinued if the slab begins to rise before the slurry is observed at the edges of the pavement. The maximum allowable lift of the slab is 3.0 mm. Pumping should also be discontinued if the slurry begins to leak on the pavement surface through a joint or crack. If the slurry appears in an unfilled hole or holes in the immediate pumping area, said hole or holes should be plugged with a tapered softwood plug.

When the voids within an area have been completely filled, the pumping should be discontinued and the hose nozzle allowed to remain in the hole for approximately 30 seconds before being withdrawn. Immediately following removal of the nozzle, a softwood plug should be driven into the hole and allowed to remain until the slurry has set. The softwood plugs should be maintained flush with the top of the pavement if traffic is to be maintained in the work area until such time as the engineer directs that the plugs be removed. Following the removal of the plug, all spillage and leakage around the hole and adjacent area should be removed. The holes should then be filled to the pavement surface with concrete repair material.

No subsealing slurry should be placed when the subgrade is frozen. The quantity of material pumped into each hole should be determined by the engineer.

The entire operation should be performed by workmen experienced in this type of work in the presence of the engineer.

Method of Measurement

The quantity to be paid for under this item should be the number of bags of portland cement incorporated in the work in accordance with the plans and specifications or as ordered by the engineer.

Basis of Payment

The unit price bid per bag of cement should include the cost of furnishing, hauling, mixing and pumping of the subsealing slurry and all labor, materials, equipment and services necessary to complete the work in accordance with the plans and specifications or as ordered by the engineer. The cost of furnishing, installing and incorporating softwood plugs, portland cement, pozzolan and testing, concrete repair material and water should be included in the price bid for this item.

Drilling holes and measuring slab movement will be paid for under separate items.

b. Inspection Control and Verification

Void filling and slabjacking problems tend to necessitate "one of a kind" grouting solutions which makes Guide Specification difficult. It is suggested that the engineer developing the specification and construction control use the preceding specifications as a guide. Also, the bidding method will depend on the amount of knowledge available on the problem; the more information on the problem and its potential solution will determine the actual bidding methods and risks to be placed on the grouting contractor. This can range from the no-risk cost plus through a lump sum method.

REFERENCES

1. ASCE Committee on Grouting (1980) "Preliminary Glossary of Terms Relating to Grouting" *Vol. 106, NoGT7, Journal of the Geotechnical Engineering Division.*

2. Glossap, R. (1960/1961). "The Invention and Development of Injection Processes". *Geotechnique*, Sept., 1960, pp 91-100, Dec., 1961, pp 225-279.

3. Houlsby, A.C. (1990). Construction and Design of Cement Grouting, *A Guide to Grouting in Rock Foundations.* John Wiley and Sons, Inc., New York, NY.

4. Karol, R.H. (1990). *Chemical Grouting*, Second Edition, Marcel Dekker, Inc., New York, NY.

5. Weaver, K. (1991). *"Dam Foundation Grouting".* American Society of Civil Engineers, New York, NY.

6. Baker, W.H. (1985), Editor. "Issues in Dam Grouting", *Proceedings of the Session Sponsored by the Geotechnical Engineering Division of ASCE in Denver, Colorado.* ASCE, New York, NY, 167 pages.

7. Kutzner, C. (1996). *Grouting of Rock and Soil.* A.A. Balkema, Rotterdam, 271 pp.

8. Henn, R.W. (1996) "Practical Guide to Grouting Underground Structures," *ASCE*, NY, NY.

9. American Society of Civil Engineers. (1982). *Grouting in Geotechnical Engineering*, Proceedings of the conference on Grouting in Geotechnical Engineering, Ed. W.H. Baker, New Orleans, LA, February 10-12, 2 Volumes.

10. Herndon, J. and Lenahan, T. (1976). Grouting in Soils, "Vol. 1 - A State of the Art Report," *FHWA-RD-76-26*, Vol. 2 - "Design and Operations Manual" *FHWA-RD-76-27*, FHWA, Washington, DC.

11. Warner, J. (1999). "Microfine Grouts." *Fundamentals of Grouting*, Short Course presented by the University of Florida, Lakewood, CO, September 12-17.

12. Tallard, G.R. and Caron, C. (1977). Chemical Grouts for Soils, "Vol. 1 - Available Materials" *FHWA-RD-77-50* and "Vol. 2 - Engineering Evaluation of Available Materials" *FHWA-RD-77-51*, FHWA, Washington, DC, Vol. 1.

13. Waller, M.J., Huck, P.J., and Baker, W.H. (1983). "Design and Control of Chemical Grouting, Vol. I - Construction Control" *FHWA-RD-82/036*; Krizek, R.I. and Baker, W.H., "Vol. 2 - Material Description Concepts" *FHWA-RD-82/037*; Baker, W.H., "Vol. 3 - Engineering Practices" *FHWA-RD-82/038*; Baker, W.H., "Vol. 4 - Executive Summary" *FHWA-RD-82/039*, FHWA, Washington, DC.

14. Bell, A.L. (1994) Editor. "Grouting in the Ground," *Proceedings of Conference of Institute of Civil Engineers, London, Nov. 1992*, Thomas Telford, London.

15. Byle, M.J. and Borden, R.H. (1995), "Verification of Geotechnical Grouting," *ASCE Committee on Grouting, ASCE Geotechnical Special Publication No. 57*, NY, NY.

16. Yonekura, R., Terashi, M., and Shibazaki, M. (1996), "Grouting and Deep Mixing," *Proceedings of IS-Tokyo-96, The Second International Conference on Ground Improvement Systems*, Tokyo, Japan, A.A. Balkema, Brookfield, VT., Vol. 2.

17. U.S. Army Corps of Engineers, (1997), "Chemical Grouting," *Technical Engineering and Design Guide*, U.S. Army Corps of Engineers, No. 24, ASCE, NY, NY.

18. American Society of Civil Engineers. (1992). *Grouting, Soil Improvement and Geosynthetics*, Proceedings of the conference sponsored by the Geotechnical Engineering Division of the American Society of Civil Engineers, Geotechnical Special Publication No. 30, New Orleans, LA, Feb. 25-28, 2 Volumes.

19. American Society of Civil Engineers. (1997). *Grouting – Compaction, Remediation, and Testing*, Proceedings of Sessions Sponsored by the Grouting Committee of the Geo-Institute of the American Society of Civil Engineers, Ed. by C. Vipulanandan, Geotechnical Special Publication No. 66, Logan UT, July 16-18.

20. American Society of Civil Engineers (1998). *Grouts and Grouting: A Potpourri of Projects.* Proceedings of Sessions of Geo-Congress 98, American Society of Civil Engineers, Geotechnical Special Publication No. 80, Boston, MA, October 18-21

20A. Warner, J. (1982). "Compaction Grouting - The First Thirty Years", *Grouting in Geotechnical Engineering*, Edited by W.H. Baker, Proceedings of Conference sponsored by Geotechnical Engineering Division, ASCE, New York, NY.

21. Baker, W.H., Cording, E.J., and MacPherson, H.H. (1983). "Compaction Grouting to Control Ground Movement During Tunneling". *Underground Space* - Vol. 7, Pergamon Press Ltd.

22. Baker, W.H. (1985). "Embankment Foundation Densification by Compaction Grouting", *Issues in Dam Grouting, Proceedings Geotechnical Engineering Division*, ASCE, April 30, 1985, Denver, CO.

23. Salley, J. R., Foreman, B., Henry, J.F., and Baker, W.H. (1987). "Compaction Grouting Test Program Pinopolis West Dam", *Soil Improvement - A Ten Year Update*, Edited by J.P. Welsh, ASCE, New York, NY, pp 245-269.

24. Henry, J.F. (1986). "Low Slump Compaction Grouting for Correction of Central Florida Sinkholes". *Proceedings of the Conference Environmental Problems in Karst Terrains and Their Solutions*, October 28-30, 1986, Bowling Green, KY, National Water Well Association, Dublin, OH.

25. Schmertmann, J., Baker, W.H., Gupta, R., and Kessler. K. (1986). "CPT/DMT QC of Ground Modification at a Power Plant". Edited by S.P. Clemence, *Proceedings of In-situ 86, Geotechnical Special Publication No. 6,* ASCE, New York, NY.

26. Welsh, J. P. and Rubright, R. (1993). "Compaction Grouting", *Ground Improvement*, edited by M.P. Moseley, Blackie Academic & Professional, Boca Raton, FL.

27. Vipulanandan, C. (1997), "Grouting: Compaction, Remediation and Testing," *Proceedings of Grouting Committee of the Geo-Institute of ASCE, Geotechnical Special Publication No. 66*, ASCE, NY, NY.

28. Bandimere, S. (1997). *Grouting: Compaction, Remediation, and Testing*, Proceedings of Sessions Sponsored by the Grouting Committee of the Geo-Institute of the American Society of Civil Engineers, Ed. by C. Vipulanandan, Geotechnical Special Publication No. 66, Logan UT, July 16-18, pp. 18-31.

29. Yahiro, T. and Yoshida, H. (1972). "Inducing Grouting Method Utilizing High Speed Water Jet". *Proceedings of 8th International Conference SM and FE*, Moscow.

30. Welsh, J.P. and Burke, G.K. (1991). "Jet Grouting - Uses for Soil Improvement", Edited by F.G. McLean, D.A. Campbell and D.W. Harris, *Geotechnical Special Publication No. 27*, ASCE, New York, NY.

31. Bell, A.L. (1993). "Jet Grouting", Edited by M.P. Moseley, *Ground Improvement*, Blackie Academic & Professional, Boca Raton, FL.

32. Xanthakos, P.P., Abramson, L.W., Bruce, D.A., (1994) "Jet Grouting, " *Ground Control and Improvement*, John Wiley & Sons, NY, NY. pp 580-679.

33. Chandler, S.C. (1997). "Lense Grouting with Fiber Admixture to Reinforce Soil," *Grouting – Compaction, Remediation, and Testing*, Proceedings of Sessions Sponsored by the Grouting Committee of the Geo-Institute of the American Society of Civil Engineers, Ed. by C. Vipulanandan, Geotechnical Special Publication No. 66, Logan UT, July 16-18, pp. 147-157.

34. Kramer, G.J.E., Tavares, P.D., and Drooff, E.R. (1994). " Settlement Protection Work for the New St. Clair River Rail Tunnel" *Proceedings of Canadian Tunneling* Bi Tech Publisher, Richmond, B.C., Canada.

35. Raabe, A.W. and Esters, K (1993). "Soil Fracturing Techniques For Terminating Settlements and Restoring Levels of Buildings and Structures", Edited by M.P. Moseley, *Ground Improvement,* Blackie Academic & Professional, Boca Raton, FL.

36. Bruce, D.A. (1989). "Methods of Overburden Drilling in Geotechnical Construction--A Generic Classification. " *Ground Engineering* 22 (7), pp. 25-32. Also published in "Drill Bits - The Official Publication of the International Drilling Federation", Fall 1989, pp. 7, 8, 110, 11, 13, 14.

37. Bruce, D.A., G.S. Littlejohn, and A. Naudts. (1997). "Grouting Materials for Ground Treatment: A Practitioner's Guide," *Grouting – Compaction, Remediation, and Testing*, Proc. of Sessions Sponsored by the Grouting Committee of the Geo-Institute of the American Society of Civil Engineers, Logan UT, Ed. by C. Vipulanandan, Geotechnical Special Publication No. 66, July 16-18, pp. 306-334.

38. Naudts, A. (1995). "Grouting to Improve Foundation Soil". *Practical Foundation Engineering Handbook.* McGraw-Hill, New York, Section 5B.

39. Bruce, D.A. (1988). "Developments in Geotechnical Construction Processes for Urban Engineering." *Civil Engineering Practice* 3 (1) Spring, pp. 49-97.

40. Baker, W.H. (1982). "Planning and Performing Structural Chemical Grouting", *Proceedings of Grouting in Geotechnical Engineering. Geotechnical Engineering Division*, ASCE, New York, NY.

41. Gularte, F.B., Taylor, G.E., and Borden, R.H. (1992) "Temporary Tunnel Excavation Support by Chemical Grouting", *Proceedings Grouting, Soil Improvements and Geosynthetics, Geotechnical Special Publication No. 30,* Edited by R.H. Borden, R.D. Holtz and I. Juran, ASCE, New York, NY, Vol. 1.

42. Byle, M.J. (1992). "Limited Compaction Grouting for Retaining Wall Repairs", *Grouting, Soil Improvement and Geosythetics, Vol. 1, Geotechnical Special Publication No. 30*, Edited by R.H. Borden, et al. Proceedings of the Conference Sponsored by the Geotechnical Engineering Division, ASCE. New York, NY.

43. Wiggins, J.H., Slossan, J.E., and Krohn, J.P. (1978). "Natural Hazards. Earthquake, Landslide, Expansive Soil". Report to *National Science Foundation Under Grants ERP-75-09998 and AEN-74-23993.*

44. Kassif, G., Livneh, M., and Wiseman, G. (1969). "Pavements on Expansive Clays". *Jerusalem Academic Press*, Jerusalem, Israel.

45. Blacklock, J.R. and Boynton, R.S. (1986), Lime Slurry Pressure Injection Bulletin, *Bulletin 331, National Lime Association*, Arlington, VA 22201.

45A. Bruce, D.A., A. Naudts, and W.G. Smoak. (1998). "High Flow Reduction in Major Structures: Materials, Principles, and Case Histories," *Grouts and Grouting: A Potpourri of Projects.* Proceedings of Sessions of Geo-Congress 98, American Society of Civil Engineers, Geotechnical Special Publication No. 80, Boston, MA, October 18-21, pp. 156-176.

46. Davidson, L. (1984). "Electronic Monitoring of Cement Pressure Grouting at Ridgeway Dam", *Bureau of Reclamation*, Denver, CO.

47. Bruce, D.A. and A. Naudts. (1998). "Contemporary Drilling and Grouting Practices for Dam Remediation," *1st Canadian Dam Association Conference*, Halifax, Nova Scotia, September 27 - October 1, 24 p.

48. Albriton, J; Jackson, L; and Bangert, R. (1984) "Foundation Grouting Practices at Corps of Engineers Dams" *Technical Report GL-84, August, 1984, U.S. Army Engineer Waterways Experiment Station, Geotechnical Laboratory*, Vicksburg, MS.

49. Moller, D.W., Minch, H.L., Welsh, J.P. (1984). "Ultrafine Cement Pressure Grouting to Control Ground Water in Fractured Granite Rock", *Innovative Cement Grouting, Edited by J.P. Welsh. American Concrete Institute*, Detroit, MI.

50. D.A. Bruce, J.A. Hamby, and J.F. Henry. (1998). "Tims Ford Dam, Tennessee: Remedial Grouting of Right Rim." *Proceedings of the Annual Conference, Dam Safety '98*, Association of State Dam Safety Officials, Las Vegas, NV, October 11-14, 13 p.

51. Okonkwo, I.O., Judd, W.R. and Altschaerfl, A.C. (1988). "A Review of Some Aspects of Grouting for Mine Subsidence Control", *Proceedings of Mine Induced Subsidence: Effects on Engineered Structures, Geotechnical Special Publication No 19*, ASCE, New York, NY.

52. Welsh, J.P. (1977). "PennDot Utilizes a New Method for Solving Scour Problems Beneath Bridge Structures", *Highway Focus, Vol. 9(1).* U.S. Department of Transportation, Federal Highway Administration, (May) pp 72-80.

53. McGregor (1967). *The Drilling of Rock.* Inst. ed. C.R. Brooks, Ltd. London, 306 pp.

54. Mitchell, J.K. (1981). "Soil Improvement - State of the Art Report," *Proc. X ICSMFE*, Stockholm, Vol. 4, pp. 509-565.

55. DePaoli, B., Bosco, B., Granata, R., and Bruce, D.A. (1992). "Fundamental Observations on Cement Based Grouts (1): Traditional Materials." *Proc. ASCE Conference, "Grouting, Soil Improvement and Geosynthetics"*, New Orleans, LA, Feb. 25-28, 2 Volumes, pp. 474-485.

56. Warner, J., N. Schmidt, J. Reed, D. Shepardson, R. Lamb, and S. Wong (1992). "Recent Advances in Compaction Grouting Technology" *Proc. ASCE Conference, "Grouting, Soil Improvement and Geosynthetics"*, New Orleans, LA, Feb. 25-28, 2 Volumes, pp.

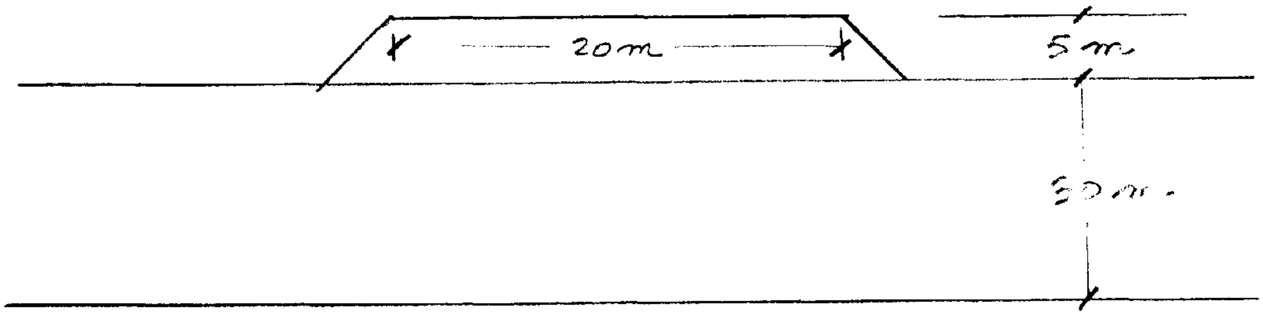
57. Committee on Grouting of the Geotechnical Engineering Division (1977). "Slabjacking - State-of-the-art", *Journal of the Geotechnical Engineering Division, Vol. GT9*, New York, NY, pp 987-1005.

WORKSHOP PROBLEM - WICK DRAINS/LT. FILL

A short 20-m wide 5-m high embankment with 2:1 side slopes in an urban area must be constructed with paving to begin 3 months after the end of construction to accommodate traffic staging. The foundation soil profile consist of 1-m of sand underlain by 30-m of soft normally consolidated clay which terminates in a dense sand and gravel deposit. Significant settlement is anticipated and an allowable settlement after paving of 0.05-m to preclude pavement distress was determined to be a performance requirement.

The designers considered two options; (a) geofoam embankment and (b) accelerated foundation consolidation by use of wick drains.

Settlement analyses indicate that 1-m of total settlement is anticipated if the embankment is constructed of normal soil borrow. Settlement will occur over a 3 year period with 95% after 30 months. The settlement under the geofoam embankment is anticipated to be proportional to the ratio of density between geofoam and that of normal soil borrow.



Determine the feasibility and cost of both options given the following:

1. The normal soil borrow density is 20KN/m^3 and its in-place cost is \$ 10/m^3.

2. The geofoam density is 1 KN/m^3 and its in-place cost is \$ 30/m^3 including the soil cover.

3. The installed cost for an equivalent 100 mm wide drain in a square pattern is \$ 2.50/lm. Use equation 1 and 2 to determine spacing.

4. The coefficient of horizontal consolidation c_h has been determined as 0.0089 m^2/day.

WORKSHOP PROBLEM DENSIFICATION

One kilometer of new highway with a base width of 35 m is to be constructed over a loose granular fill 10 m deep. The fill is characterized as loose with a blow count of (N) less than 4 and with a silt content of 5 to 10%. Groundwater is 3 m below surface.

The designers have determined that densification to 65% Relative Density or a SPT blow count of 35 to 40 is required to provide a uniform subgrade which would reduce settlement and provide sufficient strength to safely construct the embankment.

Determine the feasibility and cost of densification by dynamic compaction and vibrocompaction under the following assumptions:

1. The cost of vibrocompaction is estimated at \$ 15/lm.
2. The cost of dynamic compaction with a 16 Mg tamper is estimated at \$ 17/drop at a cc spacing of 2.5 m. Assume "n" = 0.5.

Remember that:

1 metric ton = 1 Mg = 10 kN
10 kJ/m^2 = 1 ton-meter/m^2

and

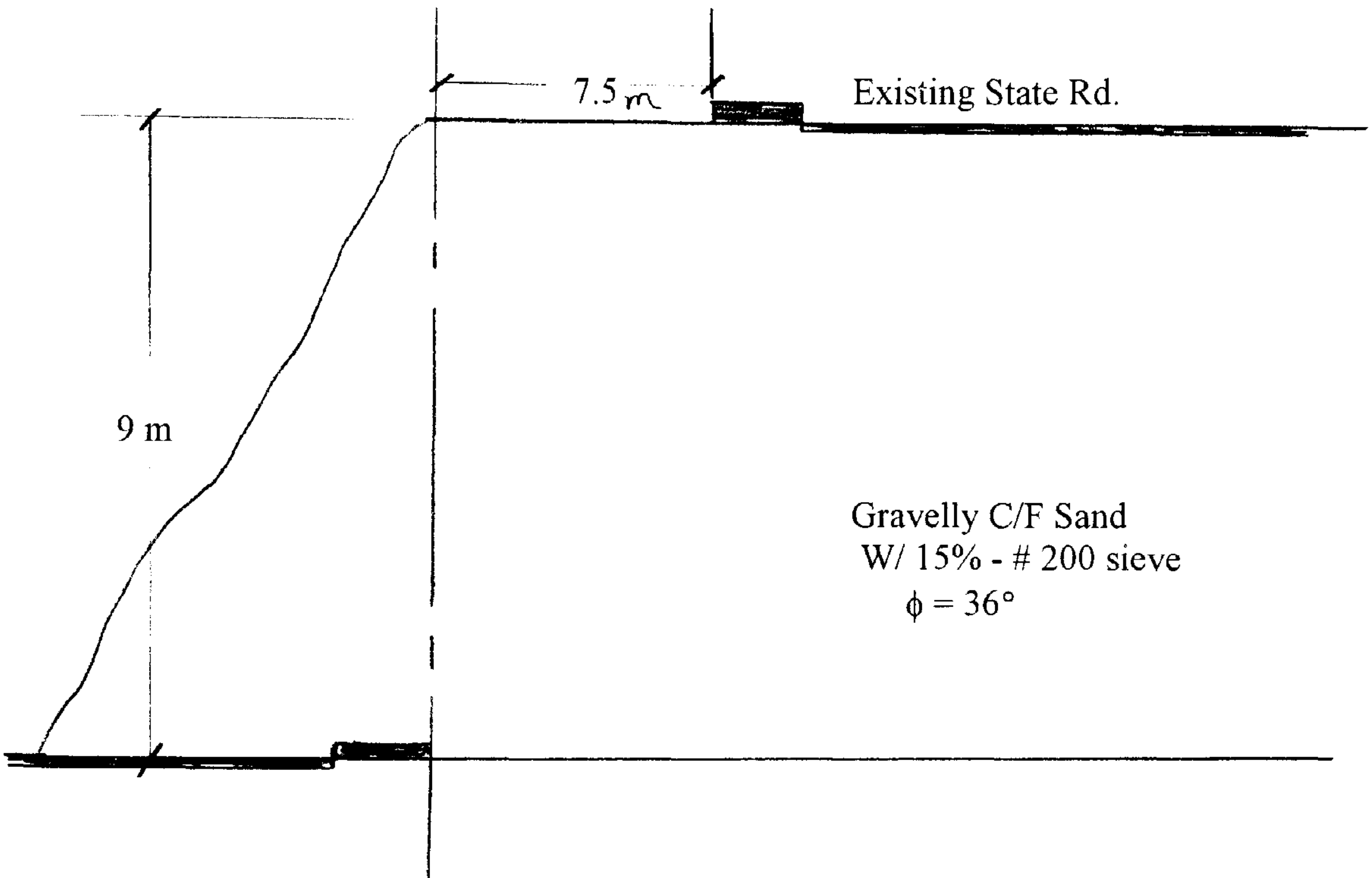
Refer to Table 3 for applied energy guidelines for dynamic compaction

- Equation 1 can be used to assess feasibility
- Equation 2 can be used to determine the number of drops per each pass

Refer to Figure 7 for Vibrocompaction design.

WORKSHOP PROBLEM - REINFORCEMENT

Determine the most cost effective type of permanent soil reinforced structure for the given geometry.

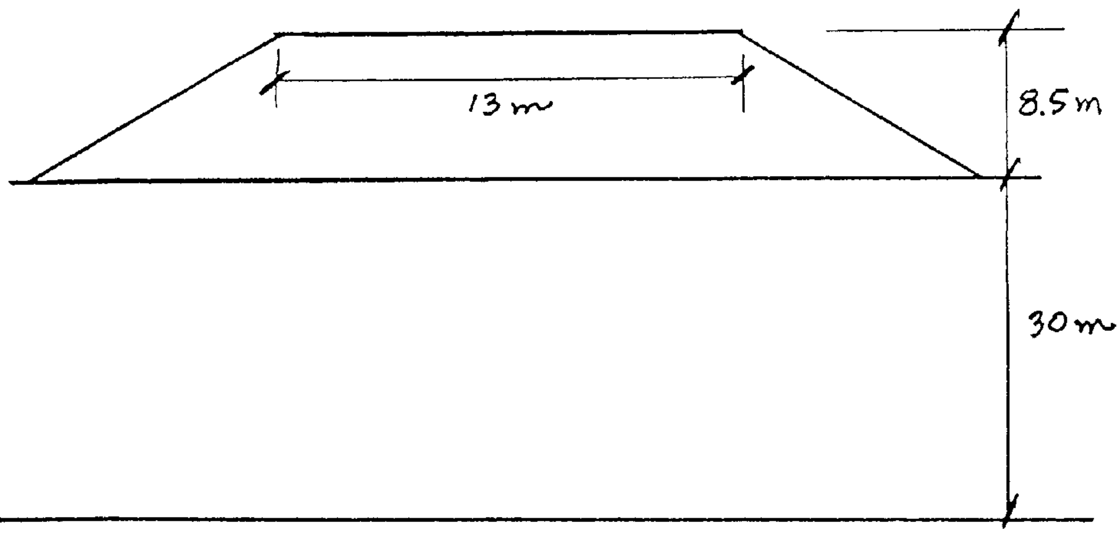


Assume:

1. Cost of permanent MSE wall materials only and erection @ \$ 200 m^2
2. Cost of permanent SN wall w/8-inch CIP facing @ \$ 600 m^2
3. Cost of temporary SN wall w/4-inch shotcrete facing @ \$ 180 m^2
4. For MSE/SN walls assume L/H = 0.7 with horizontal backfill, 0.95 with sloping surcharge. MSE embedment estimated at 1/10H.
5. Cost of excavation @ \$ 3.00 m^3
6. Cost of placing and compacting fill @ \$ 8.00 m^3

WORKSHOP PROBLEM – SOIL MIXING

An 8.5-m high, 13-m wide embankment with 2:1 side slopes must be constructed on a foundation consisting of a slightly over consolidated clay, 30-m deep. The foundation shear strength of 30 kPa (c_u) is insufficient to carry the embankment load safely and the total projected settlement and rate are unacceptable.
The designers therefore considered a lime-cement column ground improvement scheme to increase shear strength and reduce settlement.



Determine the following:

1. The stress concentration ratio, (n) if the soil Modulus is 50 c_u and the Modulus of the lime-cement column 200 c_u. The unit weight of the embankment borrow is 20 KN/m^3.
2. The area ratio (a) necessary to reduce the foundation pressure on the native soil by approximately one-half, which is necessary to sufficiently reduce settlement.
3. The spacing of the lime-cement columns when 0.80-m diameter columns are constructed.
4. The required minimum unconfined compressive strength of the column
5. The undrained shear strength of the stabilized foundation soil if the undrained shear strength of the lime-cement columns is $13c_u$.
6. The cost of lime-cement columns per lm of embankment, based on the obtained spacing and a column cost \$ 30/lm.

GROUTING PROBLEMS

1. What are the differences between the following structural soil grouting methods?

   a. Permeation Grouting
   b. Compaction Grouting
   c. Jet Grouting

2. A seismic retrofit will require improving the soil beneath an existing bridge pier to preclude liquefaction. The pier footing is 20 m long and 5 m wide. The soil profile beneath the footing is a medium to fine sand (5% silt, no clay) characterized by an average SPT blow count "N" of 15 and a porosity of 0.40 to a depth of 4 m below the footing where sound rock is found. The existing ground is 1 m above the base of the footing. To mitigate against liquefaction the designers wish to improve the foundation by grouting methods that would produce a denser foundation. Evaluate the feasibility of the following soil grouting techniques and determine a budget estimate for each on the following basis:

   a. Permeation grouting - cement grouts @ $ 0.05/liter
      - chemical grouts @ $ 0.65/liter
      - pipe drilling and installation @ $ 65/lm

   b. Compaction grouting - grout injection @ $ 50/m^3 of improved fnd.
      - pipe drilling and installation @ $ 65/lm

   c. Jet Grouting – jet grouting @ $ 200/m^3

   In addition each estimate must consider costs associated with:

   1. Mobilization/Demobilization
   2. Spacing of pipes
   3. Grout quantity
   4. Grout pipe installation costs

SOLUTION TO WICK DRAIN/LT. FILL PROBLEM

Determine if both solutions are feasible:

a. Total settlement under geofoam = 1/20 x 1m = 0.05 m
   ok - as it equals allowable settlement after paving

b. For wick drains a spacing of drains and required degree of consolidation required determines feasibility
   Degree of consolidation required ; 1m- 0.05m = 0.95/1.00 = 95%

   Determine most economical spacing with a 100mm wide drain to achieve 95% consolidation in about 90 days :

$$t = \frac{D^2}{8\,c_h}\left[\ln\left(\frac{D}{d}\right) - 0.75\right]\ln\left[\frac{1}{1-U_h}\right]$$

with $D = 1.0$ m

$$t = \frac{(1.00 \cdot 1.13)^2}{8\,(10^{-7})}\left[\ln\left(\frac{1000 \cdot 1.13}{100}\right) - 0.75\right]\ln\left[\frac{1}{1-0.95}\right]$$

$$t \approx 93 \text{ days} - \underline{\underline{\text{ok}}}$$

<u>Cost comparison</u>

Geofoam Embankment cost – 30m x 5m = 150 m^3 x \$ 30 m^3 = \$ 4500/lm

Wick Drained Embankment cost
soil borrow – 150 m^3 x \$ 10 m^3 = \$ 1500
Wick drains- 1drain per 1 m of width (40 m) plus 1 drain outside
= 1 x 42 lm x \$ 2.50 x 30m = \$ 3150
TOTAL = \$ 4650/lm

SOLUTION TO DENSIFICATION PROBLEM

a. Dynamic Compaction

Determine feasibility and cost of dynamic compaction. For feasibility check if 10 m improvement depth is attainable with conventional equipment:

$$10 \text{ m} = 0.5\,(WH)^{1/2}$$

$$WH = 400 \text{ Mgm}$$

Use a 16 Mg weight as given in problem statement.
Determine the required drop height:

$$H = \frac{400}{16} = 25 \text{ m} \qquad \text{ok feasible with 16 Mgm @ 25 m}$$

The total energy to obtain "N" > 40 is on the order of 250 KJ/m^3 since we are densifying a very loose deposit.

For 10 m of fill - $AE = 10 \text{ m} \times 250 \text{ KJ/m}^3 = 2500 \text{ KJ/m}^2$

Therefore using equation 2 for 1 drop at 2.5 m c.c. and solving for N, the number of drops:

$$2500 \text{ KJ/m}^2 = \frac{16 \text{ Mg } (25 \text{ m})\ 10 \text{ kN/Mg } (N)\ (1)}{(2.5)^2}$$

N= 3.9 use 4 drops

$$\text{Cost} = \frac{1000 \text{ m } (35 \text{ m})}{(2.5)^2} = 5600 \text{ locations } (4)\ (\$\ 17) = \$\ 380{,}800$$

b. Vibrocompaction

Feasible as it is a clean granular material.

From Figure 7b the spacing to achieve 65 % Relative Density is on the order of 2.2 m.

$$\text{Cost} = \frac{1000 \text{ m } (35 \text{ m})}{(2.2)^2} = 7231 \text{ locations } (10 \text{ m})(\$ \ 15 \text{ lm}) = \$ \ 1{,}084{,}711$$

SOLUTION TO REINFORCEMENT PROBLEM

Option 1 - MSE wall with precast concrete facing at edge of shoulder with a 2:1 surcharge slope and a temporary SN wall at end of reinforcement. Leave a 1 m wide shoulder for guardrails. L/H for MSE wall @ 0.95H to include 1/10 H embedment; therefore H = 6.6 m and L = 6.27 or 6.5 m (reinforcement supplied on 0.5 m increments).
Construction is therefore feasible.

Cost per 1m as follows:

a.	MSE wall required = 6 m + 1/10 (6) = 6.6m		
		$6.6\ m^2$ x \$ $200/m^2$	= \$ 1,320
b.	SN temporary wall	$9\ m^2$ x \$ $180/m^2$	= \$ 1,620
c.	Excavation and backfill quantities	= 6.5m x 9.6 m	= $63\ m^3$
d.	Excavation	$63\ m^3$ x \$ $3.00/m^3$	= \$ 189
e.	Backfill	$54\ m^3$ x \$ $8.00/m^3$	= \$ 432
		TOTAL	= \$ 3,561

Option 2 - Permanent SN wall at shoulder, with 2:1 Surcharge slope is feasible.

Cost per lm as follows:

$6\ m^2$ x \$ $600/m^2$	= \$ 3,600
TOTAL	= \$ 3,600

SOLUTION TO SOIL MIXING PROBLEM

1. The stress concentration ratio can be estimated as a first approximation as the ratio of the Modulus of the lime-cement to the Modulus of the foundation soil.

$$n = \frac{M_{lime\text{-}cement}}{M_{soil}} = \frac{200}{50} = 4$$

2. (a) Determine the embankment stress "q" on the foundation:

$$8.5\text{m} \times 20\ \text{KN/m}^3 = 170\ \text{KN/m}^2$$

(b) Determine an area ratio that would reduce the stress on the native foundation soil to approximately 85 kPa (one-half of "q"):

$$\sigma_c = \frac{q}{[\ 1 + (n-1)\ a_s\ ]} = \frac{170}{[\ 1 + (4-1)\ a_s\ ]}$$

with $a_s = 0.333$ $\quad \sigma_c = 85$ kPa *ok*

3. With an area ratio of 0.333 the spacing, D, is:

$$D^2 = \frac{A_{col}}{a_s} = \frac{0.785(0.80)^2}{0.333} \quad D = 1.228\ \text{m; say } 1.3\text{m}$$

4. The minimum required unconfined compressive strength is equal to the stress on the column times a factor of safety – say 2

$$\sigma_s = \frac{nq}{[1 + (n-1)\ a_s]} = \frac{4(170)}{[1 + (4-1)\ 0.33]} = 340\ \text{kPa}$$

The minimum required unconfined compressive strength is therefore 2 (340) = 680 kPa *ok* - as the undrained shear strength of the column was given as $13c_u$ or 13(30 kPa)= 390 kPa (2) = 780 kPa u.c. strength

5. The undrained shear strength of the improved soil mass is:

$$\tau = \tau_{column}\,(a_s) + c_u\,(1\text{-}a_s) = (13\ x\ 30)\ 0.333 + 30\ (1\text{-}0.333)$$

$$\tau = 130 + 20 = 150 \text{ kPa}$$

6. Cost of lime-cement columns per lm of embankment:

1/1.3 *x* 47/1.3 = 27.81 columns/lm *x* 30m *x* \$ 30/lm = \$ 25,029 lm

GROUTING PROBLEM SOLUTIONS

1. What are the differences between the listed soil grouting methods?

Permeation Grouting – Permeation grouting either with cement or chemical grouts is used to increase cohesion and/or reduce permeability of in situ granular soils. The type of grout used will depend on the grain size of the in situ soil and the desired results.

Compaction Grouting – Compaction grouting is the injection of low slump grout under high pressure. This technique is valuable in densifying in situ soils, underpinning and for settlement control especially in connection with soft ground tunneling.

Jet Grouting – Jet grouting employs high pressure erosive jets of water or grout to break down the soil structure, removing varying proportions of soil and replacing them with a cement based grout. Soil not removed becomes mixed with the grout in situ to form a treated mass. This grouting technology is the most versatile as it can treat soils ranging from silt to gravel.

2. Liquefaction Mitigation Estimates

a. Permeation Grouting. The foundation sands are too fine to accept cement slurry grouting, but are ideal for chemical grouting.

1. Calculate volume of chemical grout required beneath the pier plus 45 degree cone outside the pier:

$$20 \times 5 \times 4 + \frac{20 \times 4 \times 4 \times 2}{2} + \frac{5 \times 4 \times 4 \times 2}{2} = 800 \text{ m}^3$$

volume of chemical grout = 800 m^3 x 0.40 x 1000 = 320,000 liters
Cost of grout = 320,000 x \$ 0.65/l = \$ 208,000

2. Grout pipes required – use 1.3 m spacing; under 5 m width use 10 rows of which 4 are drilled through the pier. Under the 20 m length plus 45 degree cone use 22 rows (28/1.3).
Therefore 10 x 22 = 220 pipes are required.
Drilling depth is 4m + 1m = 5m x 220 = 1,100 m
Cost is 1,100 x \$ 65/lm = \$ 71,500

3. Budget cost – Mobilization - \$ 40,000

Budget cost –	Mobilization	- \$ 40,000
	Chemical grout	- \$ 208,000
	Pipe Installation	- \$ 71,500
Budget		- \$ 319,500

b. Compaction Grouting. Use 1.5 m spacing as tighter spacing is required to increase density of in situ soils within the 800 m^3 volume previously calculated.

1. Based on 1.5m spacing 19 rows by 9 rows would be required for a total of 171 pipes.
2. Grout cost = 800 m^3 x \$ 50 m^3 = \$ 40,000
3. Required drilling = 171 x 5 m = 855 lm x \$ 65/lm = \$ 55,575
4. Mobilization/Demobilization = \$ 25,000

Budget = \$ 120,575

c. Jet Grouting. The foundation is in sand, therefore it is conceptually feasible with double rod system. Work must be carefully sequenced to preclude loss of support under the footing during jet grouting operations.

1. Volume grouted is on the order of 800 m^3 which can be accommodated by a variety of column diameters and center to center spacing. The planned cc spacing should be less than the column diameter to ensure overlap.

Jet grouting cost –	800 x \$ 200/m^3	= \$ 160,000
Mobilization/Demobilization		= \$ 40,000
Budget		= \$ 200,000

GROUND IMPROVEMENT METHODS/APPLICATIONS

METHODS	APPLICATIONS/FUNCTIONS
1. WICK DRAINS	ACCELERATES FOUNDATION CONSOLIDATION
2. LIGHTWEIGHT FILL	REDUCES LOAD ON FOUNDATION – REDUCES SETTLEMENT
3. VIBRO COMPACTION	DENSIFIES LOOSE SANDS – INCREASES BEARING CAPACITY DECREASES SETTLEMENT
4. DYNAMIC COMPACTION	DENSIFIES LOOSE GRANULAR/DEBRIS FOUNDATIONS INCREASES BEARING CAPACITY DECREASES SETTLEMENT
5. MSE WALLS RS SLOPES	FILL RETAINING STRUCTURES
6. SOIL NAILING	IN- SITU CUT RETAINING STRUCTURES
7. STONE COLUMNS	REINFORCES FOUNDATION SOFT CLAYS/SILTS - INCREASES SHEAR STRENGTH – ACCELERATES SETTLEMENT

8. SOIL MIXING	IN-SITU PHYSICO-CHEMICAL ALTERATION OF FOUNDATION SOILS TO INCREASE THEIR TENSILE/COMPRESSIVE STRENGTH
9. GROUTING	DENSIFIES BY COMPACTION GROUTING – REPLACES BY JET GROUTING – ALTERS BY PERMEATION GROUTING – FILLS VOIDS BY BULK FILLING

DESIGN – CONTRACTING - SUMMARY

G.I. TYPE	DESIGN METHOD	SPECIFICATION TYPE
WICK DRAINS -	*RIGOROUS*	*METHOD*
LIGHT. FILLS -	*RIGOROUS*	*METHOD*
DYNAMIC COM. -	*EMPIRICAL*	*METHOD OR PERFORMANCE*
VIBROCOM. -	*EMPIRICAL*	*PERFORMANCE*
MSE -	*EMP./RIG.*	*METHOD/PERF.*
RSS	*RIGOROUS*	*METHOD*
SOIL NAILING	*EMP./RIG.*	*METHOD/PERF.*
STONE COLUMN -	*EMPIRICAL*	*METHOD OR PERFORMANCE*
SOIL MIXING -	*EMPIRICAL*	*PERFORMANCE*
GROUTING -	*EMPIRICAL*	*PERFORMANCE*

COMPARATIVE COSTS

METHOD	UNIT COST	COST OF TREATED VOLUME ($/M^3)
WICK DRAINS	$ 1.50-4.00/m	$ 0.80-1.60
LIGHT. FILL		
w/Granular	$ 3.00-21.00/m^3	
w/Tires-Wood	$ 12.00-30.00/m^3	
w/Geofoam	$ 35.00-65.00/m^3	
w/Foam Concrete	$ 65.00-95.00/m^3	
DYNAMIC COMP.	$ 6.00-11.00/m^2	$ 1.00-2.00
VIBROCOMP.	$ 15.00-25.00/m	$ 1.00-4.00
MSE	$160.00-300.00/m^2	
RSS	$110.00-260.00/m^2	
SOIL NAILING	$400.00-600.00/m^2	
STONE COLUMN	$ 40.00-60.00/m	$ 50-75
SOIL MIXING		
W/ CEMENT		$ 60-150
W/LIME-CEMENT	$ 30.00/m	$ 60
GROUTING		
PERMEATION	$65.00/m + $0.70/L	
COMPACTION		$30-200
JET		$200-275

IMPEDIMENTS TO ACCEPTANCE FOR GROUND IMPROVEMENT TECHNOLOGIES

1. FOR SOME TECHNOLOGIES, LIMITED POTENTIAL APPLICATIONS PRECLUDES DEVELOPING EXPERTISE

2. EMPIRICAL DESIGN BASIS SOMETIMES NOT WELL DOCUMENTED

3. RISK PERCEPTION

4. REQUIRES CONTRACTORS WITH SPECIAL EQUIPMENT

5. OFTEN BASED ON PROPRIATARY EQUIPMENT NOT LOCALLY AVAILABLE

6. WITH PERFORMANCE BASED SPECIFICATIONS, CLEAR AND OBTAINABLE PERFORMANCE BENCHMARKS MUST BE ESTASBLISHED. THIS IS ATYPICAL IN PUBLIC SECTOR CONTRACTING

7. SCARCITY OF RELIABLE COST DATA

8. INERTIA

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