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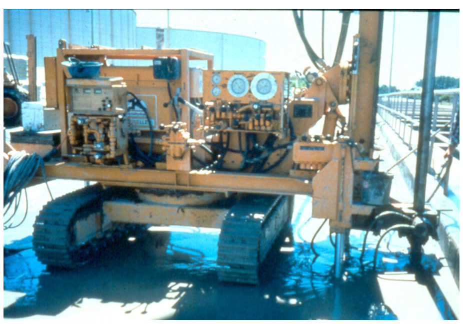
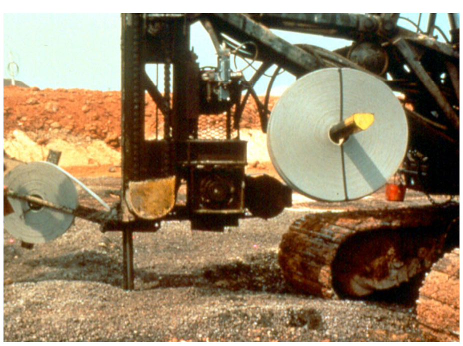
NHI Course No. 132034

Ground Improvement Methods

Reference Manual – VOLUME II

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16. ABSTRACT This is the reference manual for FHWA NHI course No. 132034 on Ground Improvement Techniques and contains eleven stand alone Technical Summaries on specific ground improvement methods. The summaries provide current practice for design, construction, contracting and quality procedures for the respective technologies. The individual summaries are intended to serve as a primer on a particular subject; and are not intended to present a comprehensive or complete thesis on the technology. Companion, comprehensive technical references for each technology are provided.		
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SI CONVERSION FACTORS				
APPROXIMATE CONVERSIONS FROM SI UNITS				
Symbol	**When You Know**	**Multiply By**	**To Find**	**Symbol**
		LENGTH		
mm	millimeters	0.039	inches	in
m	meters	3.28	feet	ft
m	meters	1.09	yards	yd
km	kilometers	0.621	miles	mi
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m^2	square meters	10.764	square feet	ft^2
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m^3	cubic meters	1.307	cubic yards	yd^3
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kg	kilograms	2.202	pounds	lb
tonnes	tonnes	1.103	tons	tons
		TEMPERATURE		
°C	Celsius	1.8 C + 32	Fahrenheit	°F
		WEIGHT DENSITY		
kN/m^3	kilonewton / cubic meter	6.36	poundforce / cubic foot	pcf
	FORCE and PRESSURE or STRESS			
N	newtons	0.225	poundforce	lbf
kN	kilonewtons	225	poundforce	lbf
kPa	kilopascals	0.145	poundforce / square inch	psi
kPa	kilopascals	20.9	poundforce / square foot	psf

PREFACE

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

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

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

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

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

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

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

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

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

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

Dr. Donald Bruce	- ECO Geosystems, Inc.
Dr. James Collin	- The Collin Group
Mr. Albert DiMillio	- FHWA
Mr. Richard Endres	- Michigan DOT
Mr. John Hooks	- FHWA
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

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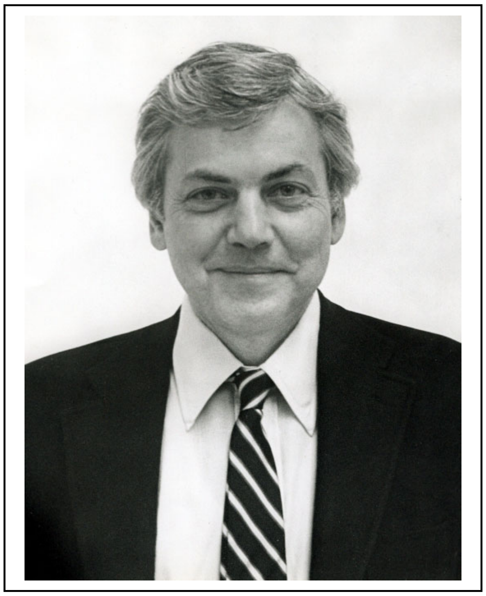

This manual is the last major work of the lead author, **VICTOR ELIAS, P.E.**

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

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

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

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

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

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

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

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

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

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

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

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

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

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

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

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

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

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

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

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INTRODUCTION

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

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

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

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

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

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

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

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

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

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

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

5. *Preliminary Selection*. Preliminary selection of potentially applicable method(s) is generally made on a qualitative basis taking into consideration the performance criteria, limitations imposed by subsurface conditions, schedule and environmental constraints and the level of improvement that is required.

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

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

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

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

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

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

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

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

Table 2.a Comparative Costs (SI Units)

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

Notes:

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

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

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

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

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

Notes:

a. Usually add mobilization charge of \$8,000 to \$10,000, as well as the cost of drainage blanket and instrumentation.

b. \$15/lm when no backfill is placed around the probe and \$25/lm when granular backfill is added. In addition to unit cost, a mobilization/demobilization cost of \$15,000 per rig should be assumed.

c. The minimum cost for vibro-replacement stone column installation, based on readily available suitable backfill material, is \$14/lft., with a dry vibro displacement stone column starting at \$18/lft. In addition to unit cost, a minimum mobilization/demobilization cost of \$15,000 per rig should be assumed.

d. Add mobilization/demobilization charge of \$10,000 to \$50,000.

Technical Summary # 7:

COLUMN SUPPORTED EMBANKMENTS

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CHAPTER 1
DESCRIPTION AND HISTORY

The problems associated with constructing highway embankments over soft compressible soil (*i.e.,* large settlements, embankment stability, and the long period of time required for consolidation of the foundation soil) have lead to the development and/or extensive use of many of the ground improvement techniques in use. Wick drains, surcharge loading, geosynthetic reinforcement, stone columns, deep soil mixing, and vibro-concrete columns have all been used to solve the settlement and embankment stability issues associated with construction on marginal soils. However, when time constraints are critical to the success of the project, owners have resorted to another innovative approach: column supported embankments (CSE) reinforced with geosynthetic reinforcement. In the last 15 years, this technology has been used successfully on several projects both in the United States and abroad.

1.1 DESCRIPTION

Column supported embankments consist of vertical columns that are designed to transfer the load of the embankment through the soft compressible soil layer to a firm foundation. The selection of the type of column used for the CSE will depend on the design loads, constructability of the column, cost, etc., and will be discussed in more detail in Chapters 2 and 3. The load from the embankment must be effectively transferred to the columns to prevent punching of the columns through the embankment fill causing differential settlement at the surface of the embankment. If the columns are placed close enough together, soil arching will occur and the load will be transferred to the columns. Figure 1 shows a conventional CSE. The columns are spaced relatively close together, and some battered columns are required at the sides of the embankment to prevent lateral spreading. In order to minimize the number of columns required to support the embankment and increase the efficiency of the design, a geosynthetically reinforced load transfer platform (LTP) may be used. The load transfer platform consists of one or more layers of geosynthetic reinforcement placed between the top of the columns and the bottom of the embankment. Figure 2 shows schematically a CSE with geosynthetic reinforcement.

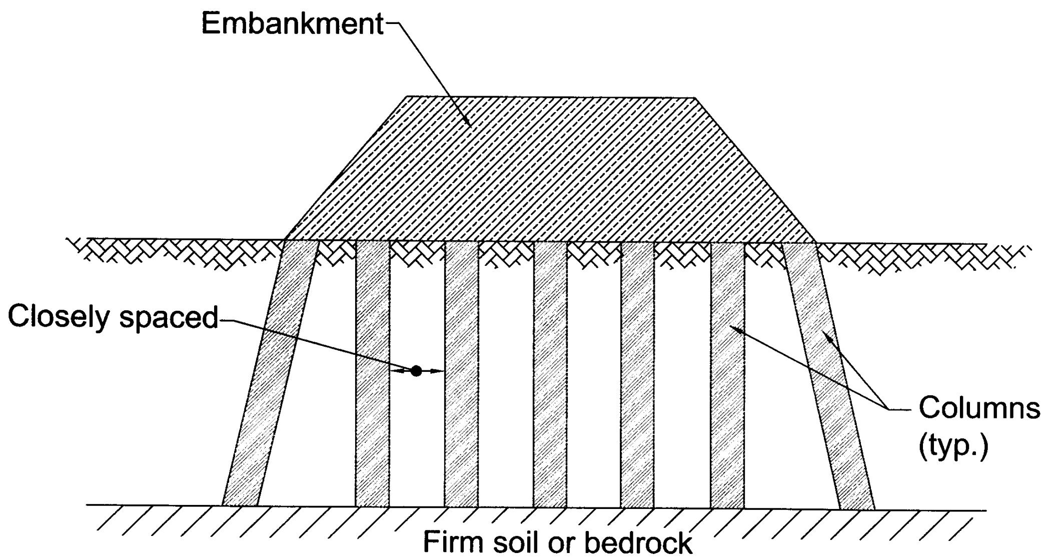

Figure 1. Conventional Column Supported Embankment.[1]

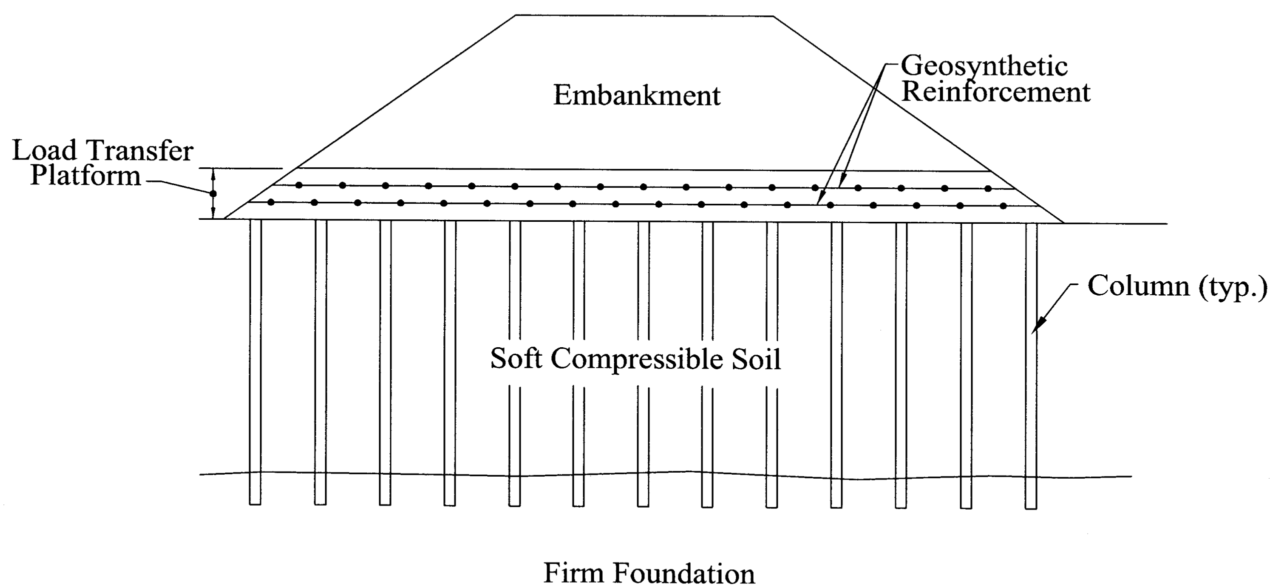

Figure 2. Column Supported Embankment with Geosynthetic Reinforcement.[1]

1.2 FOCUS AND SCOPE

The focus and scope of this technical summary on geosynthetic reinforced CSE is to identify problems that have been successfully solved by the use of CSE and to synthesize the current state-of-the-practice of CSE construction and design. In addition, this technical summary will provide guidance on the selection process for CSE. 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 CSE can solve a specific problem. It is the intent of this document to serve as a reference on CSEs and how they may solve a specific problem by discussing their construction, utilization, and limitations.

1.3 HISTORICAL OVERVIEW

The first documented use of CSE with geosynthetic reinforcement was in 1984 for a bridge approach embankment in Europe.(2) Concrete piles were used as the columns for the project. Each column had a reinforced concrete pile cap. The clear span between pile caps varied from 2-3 meters (6.6-10 ft.). One layer of geosynthetic reinforcement was used to create the load transfer platform. The height of the embankment was 9 m (30 ft.).

The first application of CSE with geosynthetic reinforcement in the United States was in 1994 for the Westway Terminal in Philadelphia, PA. This project involved the support of a large diameter tank for the storage of molasses. The foundation consisted of vibro-concrete columns and a LTP, and is shown in Figure 3. The platform consisted of a well graded granular fill, reinforced with three layers of geogrid reinforcement. The CSE was selected over a more conventional pile foundation with a concrete mat because of both time and money savings.

One of the first (2001) transportation related projects in the United States to use CSE was for an embankment over soft soils, at a river crossing, for the New Jersey Light Rail.(3) The foundation for the embankment consisted of vibro-concrete columns and an LTP. The VCCs were placed on a 2.3-3.0 m (6.6-10 ft.) center-to-center triangular spacing. The platform was 1 m (3 ft.) thick, and was reinforced with three layers of geogrid. A well-graded granular soil was used as structural fill for the platform. Figure 4 shows a typical cross-section of the project. The CSE was selected for this project to eliminate the "bump" at the end of the bridge without having to wait for the foundation soil to consolidate.

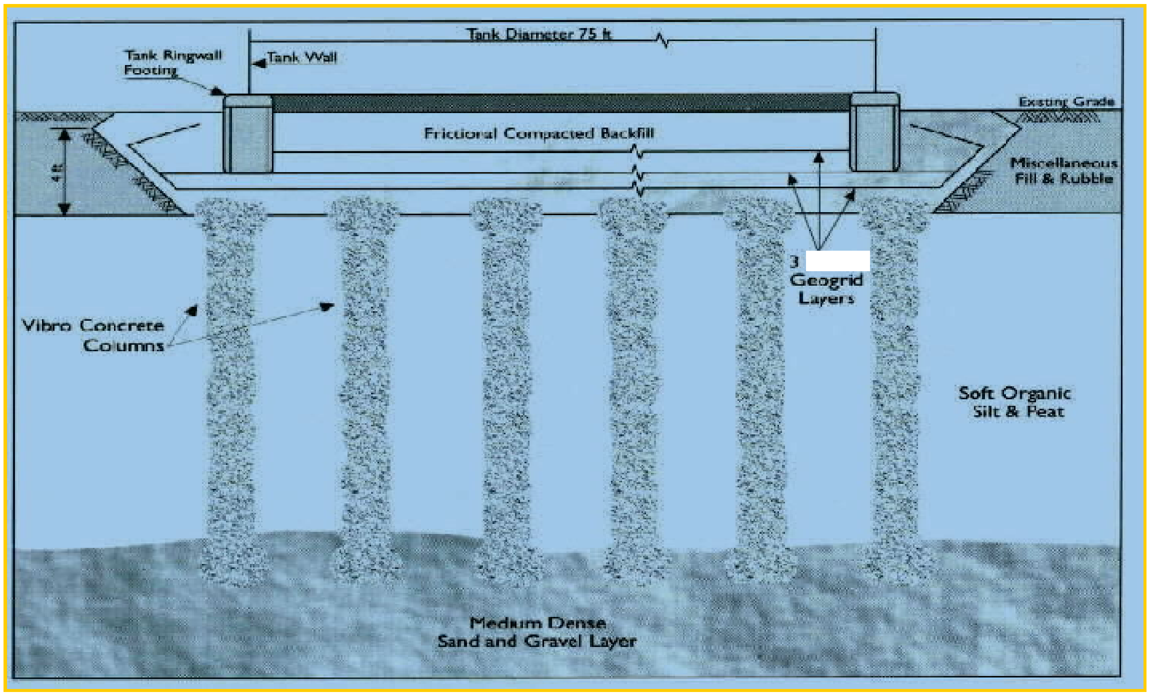

Figure 3. Westway Terminal Project.

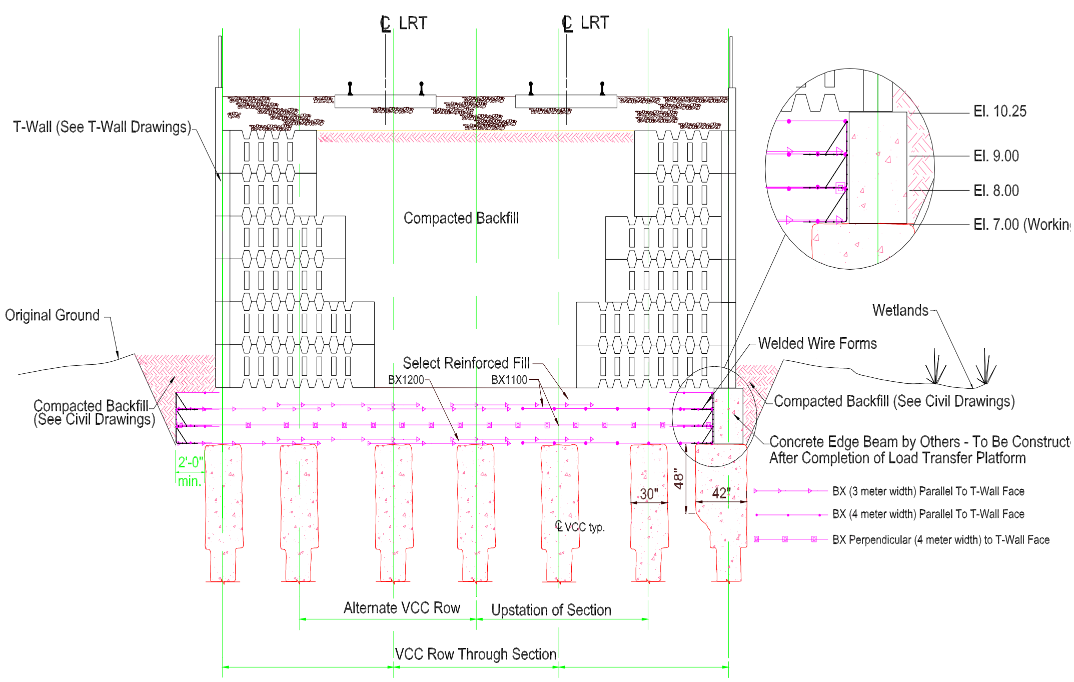

Figure 4. New Jersey Light Rail Project.[3]

The use of CSE with geosynthetic reinforcement has increased dramatically in the last decade both in the United States and abroad. More than 20 case histories are now available in the literature documenting the use of this technology.[4]

1.4 TYPES OF COLUMNS

There is a wide range of columns that may be used for CSE. Conventional (*i.e.,* timber, steel H, steel pipe, pre-cast concrete, and cast in-place concrete shell) piles may be used for pile supported embankments. However, conventional piles, with the exception of timber piles, have a rather high structural capacity (*i.e.,* 400-2000 kN (90-450 kips)) that is seldom required for CSE and are, therefore, economically not as attractive as non-traditional columns. Augered piles have been used in Europe with success.

The newer elements that have been used for columns in CSEs include soil mix columns; Stone Columns, Geotextile Encased Columns (GEC), Geopier Rammed Aggregate Piers, and Vibro-Concrete Columns (VCC). These columns are discussed in the Soil Mixing and Stone Column Technical Summaries. Combined soil stabilization (CSV) and AU-Geo piles are briefly introduced later in this Technical Summary.

CSV is a new technology that has recently been brought to the United States from Germany. The columns are constructed by introducing dry granular material in the soft foundation soil by an auger, which has a compaction head attached at its tip. The auger rotates in the opposite direction to the pitch of the flights. Soil is, therefore, not removed during the drilling operation, but rather compacted around the auger. As the auger advances into the ground, it compacts the soil around the auger. During withdrawal of the auger, dry granular material (typically a sand cement mix) is transported to the tip of the auger and compacted. CSV columns have typical diameters of 150-200 mm (6-8 in.) and have capacity of 45-90 kN (10-20 kips).

AU-Geo is a new piling system that was developed in Holland. This system consists of a PVC pipe that is installed through a mandrel through the soft foundation soil into a firm bearing layer. An enlarger plate is attached to the bottom of the PVC pipe to increase the end bearing resistance of the column. A 220-mm (8½-in.) diameter steel pipe is driven into the ground to the required bearing depth. Inside the mandrel is a 15-mm (¾ -in.) PVC pipe, with an end plate that is just smaller than the inside diameter of the mandrel. The mandrel is removed, leaving the PVC pipe in-place. The PVC pipe is filled with concrete. A 300 mm by 300 mm (12 in. by 12 in.) pile cap is typically attached to the pile. The maximum capacity of the AU-Geo pile is 150 kN (34 kips).

1.5 TYPES OF LOAD TRANSFER PLATFORMS

The LTP transfers the embankment load to the columns. Two types of load transfer platforms are available. A reinforced concrete structural mat may be used to transfer the embankment load to the columns. This requires a structural design of the mat to assure that the load is effectively transferred to the columns. Concrete mats have generally been found to be economically cost prohibitive and will not be discussed further in this Technical Summary.

The second type of LTP consists of one or more layers of geosynthetic reinforcement and select backfill to create a system to transfer the embankment load to the foundation columns, as shown in Figure 2. The remainder of this Technical Summary will focus on the design and construction of the geosynthetic reinforced LTP.

CHAPTER 2
DESIGN CONSIDERATIONS

This chapter discusses applications, advantages and disadvantages, and design considerations for CSE technology.

2.1 APPLICATIONS

CSE have traditionally been used to support embankments over soft soil when time is not available to allow consolidation of the soft foundation soil when using wick drains and surcharge loads, or when differential or total settlement and overall stability are a concern. The main purpose of a CSE is to transfer the embankment loads through the columns to a competent soil or rock layer beneath the soft foundation soil. Applications where CSE technology is appropriate for transportation include

- embankment stabilization
- roadway widening
- bridge approach fill stabilization
- bridge abutment and other foundation support

Other applications that have utilized this technology include foundation support for storage tanks, commercial office building foundation support (*i.e.,* shallow foundations supported on CSE), and retaining wall foundation support. The database of successful projects continues to expand, and with the development of new, more cost effective column systems, CSE use will continue to grow.

One typical application of CSE technology is the stabilization of large area loads, such as highway embankments. The use of CSEs offers a practical alternative, where conventional embankments cannot be constructed due to stability, time, or environmental considerations. Applications include moderate to high fills on soft soils, and embankment fills that may be contained by Mechanically Stabilized Earth (MSE) retaining walls.

A considerable amount of highway widening and reconstruction work will be required in future years. Some of this work will involve building additional lanes immediately adjacent to existing highways constructed on moderate to high fills over soft cohesive soils, such as those found in wetland areas. For this application, differential settlement between the existing and new construction is an important consideration, in addition to embankment stability. Support of the new fill on CSE offers a viable design alternative to conventional construction.

CSE 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 2001, the New Jersey Light Rail used a column supported embankment for the approach embankment for a river crossing. One side of the embankment was contained by a modular concrete retaining wall system, and the other side of the embankment sloped downward to the adjacent grade. The CSE included the use of VCC as the columns and three layers of geosynthetic reinforcement to create the LTP (Young, et al., 2003) to eliminate the "bump" at the end of the bridge.

Under favorable conditions, CSE 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. Embankment fills can be placed more quickly, due to the fact that the embankment places little or no load on the soft foundation soil.

CSE 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 CSE.

CSE have been used successfully to support building foundations when located in areas that contain soft compressible foundation soils.

2.2 ADVANTAGES AND DISADVANTAGES OF CSE

Advantages

CSE provide a technical and potentially economical alternative to more conventional construction techniques (*i.e.,* surcharge loading and wick drains, staged construction with or without geosynthetic reinforcement). The key advantage to CSE is that construction may proceed rapidly in one stage. There is no waiting time for dissipation of pore water pressure in the soft foundation soil. CSE 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, the type of column used for the CSE system may be selected to minimize or eliminate the potential for vibrations. Total and differential settlement of the embankment may be drastically reduced when using CSE over conventional approaches.

One major benefit of CSE technology is that it is not limited to any one column type. If contaminated soils are anticipated at a site, the column type may be selected so that there are no spoils from the installation process. If very soft soil is anticipated, VCC, GEC, augered piles or timber piles may be selected as column type for the project. In stronger foundation soils, stone columns or rammed aggregate piers may be economically more attractive. The designer has the flexibility of selection of the most appropriate column for the project.

Disadvantages

A major disadvantage of CSE is often initial construction cost when compared to other solutions. However, if the time savings when using CSE technology is included in the economic analysis, the cost may be far less than other solutions.

Another major disadvantage is that there is currently no single well-accepted design procedure. There are many different design approaches, and they all give different results. Without some standardization of the LTP design, the technology will be limited in its use and acceptance.

2.3 FEASIBILITY EVALUATIONS

CSE may be used whenever an embankment must be constructed on soft compressible soil. To date, the technology has been limited to embankment heights in the range of 10 meters (33 ft). The depth of the soft soil layer is not a critical component in the determination of feasibility because of the many different types of columns available for use.

A generalized summary of the factors that should be considered when assessing the feasibility of utilizing CSE technology on a project is presented below:

1. The preliminary spacing of the columns should be limited so that the area replacement ratio (***i.e.,*** ratio of the cross-sectional area of the column to the cross-sectional area of the area of influence for each column (see Section 4.2)) is between 10-20%. This recommendation is based on the empirical performance of documented case histories of CSE [(4)].

2. The clear span between columns should be less than the embankment height and should not exceed 3 m (10 ft.). This requirement is based on documented case histories. Wider clear spans may lead to unacceptable differential settlement between columns.

3. The fill required to create the LTP shall be a select structural fill with an effective friction angle greater than or equal to 35°.

4. The columns shall be designed to carry the entire load of the embankment.

5. CSE technology reduces post construction settlements of the embankment surface to typically less than 50-100 mm (2-4 in.).

2.4 ENVIRONMENTAL CONSIDERATIONS

The selection of the most appropriate column system should consider the environmental effects of the installation. For example, if stone columns were being considered for a project, vibro-replacement stone columns are traditionally 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. The designer may select an alternate column system that does not replace the in-situ soils (*i.e.,* dry vibro-displacement stone columns, GEC, VCC, etc.).

In urban environments where noise and vibrations may be unacceptable, appropriate columns may be selected accordingly.

2.5 ALTERNATIVE IMPROVEMENT METHODS

Alternate ground improvement systems that should be considered when evaluating CSE include surcharge loading with or without wick drains, staged construction with or without geosynthetic reinforcement, and lightweight fill. The Technical Summaries for Wick Drains, Lightweight Fill, and MSE Walls and Reinforced Slopes should be reviewed for more information on these alternate systems. In addition to alternate ground improvement systems, designers should also consider using a bridge structure when constructing embankments on soft compressible soils.

CHAPTER 3
CONSTRUCTION MATERIALS AND EQUIPMENT

3.1 LTP MATERIALS

Geosynthetic Reinforcement

The geosynthetic reinforcement material used to create the load transfer platform has typically been either a single layer of high strength geotextile or geogrid, or several layers of low strength biaxial geogrid. The type and strength of the geosynthetic reinforcement is a function of the design model used for analysis of the LTP (*i.e.,* catenary or beam (see Chapter 4)), spacing between columns, and height of embankment. Many designers require that a cushion layer of fill be placed between the top of the columns and the geosynthetic reinforcement. The primary function of this layer of fill is to eliminate abrasion occurring between the top of the column and the reinforcement. The geosynthetic reinforcement should be rolled out in the direction indicated on the construction drawings. All wrinkles and slack should be removed prior to fill placement. During fill placement, no construction equipment should be allowed to travel directly on the reinforcement. A minimum of 150 mm (6 in.) of fill should be placed between the reinforcement and any construction equipment.

The requirements for seams shall be considered in the design and the selection of the geosynthetic reinforcement. LTPs constructed to date have used both sewn seams and overlap seams; however, the type of seam should be considered in the design of the LTP.

Backfill Material

The backfill material used to create the LTP is a critical component of the system. Arching in the soil above the columns is considered an integral component in the transfer of stress from the embankment to the columns. It is, therefore, important that the soils in the zone where the arch is formed be frictional material with high shear strength. Well graded granular fill is considered the most ideal material for constructing the platform. Above the platform, a non-select fill may be used to construct the remainder of the embankment.

3.2 COLUMNS

The columns are an integral part of CSE, and many types of columns are available to the designer. Driven piles (*i.e.,* timber, steel H, steel pipe, pre-cast concrete, cast-in-place concrete shell, and shells driven without mandrel) may be used. Driven piles are generally considered to be very stiff columns with a modulus of elasticity between 7,000-210,000 MPa (1,000-30,000 ksi) (modulus for timber piles is 7,000 MPa). The load carrying capacity of driven piles may be calculated in accordance with FHWA HI-97-013[5]. Augered piles and minipiles are typically drilled piles and are also considered to be stiff columns. Settlement of these types of columns is typically governed by the capacity of the foundation soil.

Stone columns and rammed aggregate piers (*i.e.,* Geopiers) are columns that have modulus values between 30-60 MPa (5-9 ksi), which is considerably lower than driven piles. The design of these columns is presented in the Stone Column Technical Summary. VCC are considered a sister technology to stone columns, with concrete replacing the stone in the column. VCC are, therefore, considerably stiffer than stone columns. For more information on VCC, see the Stone Column Technical Summary.

GEC, CSV, and AU-Geo columns are the newest column systems available and, like stone columns and rammed aggregate piers, are not as stiff as driven piles.

Table 1 lists the types of columns that may be used for CSE and some of their important characteristics.

3.3 COLUMN CAPS

Column caps have been used in Europe to decrease the clear span between columns. Figure 5 shows a CSE with geosynthetic LTP and column caps. The caps usually consist of cast-in-place concrete. Reinforcing steel may be required. Currently there is little information on the design of column caps. Design issues for column caps are focused on the connection between column and cap, with respect to lateral loads and bending moments.

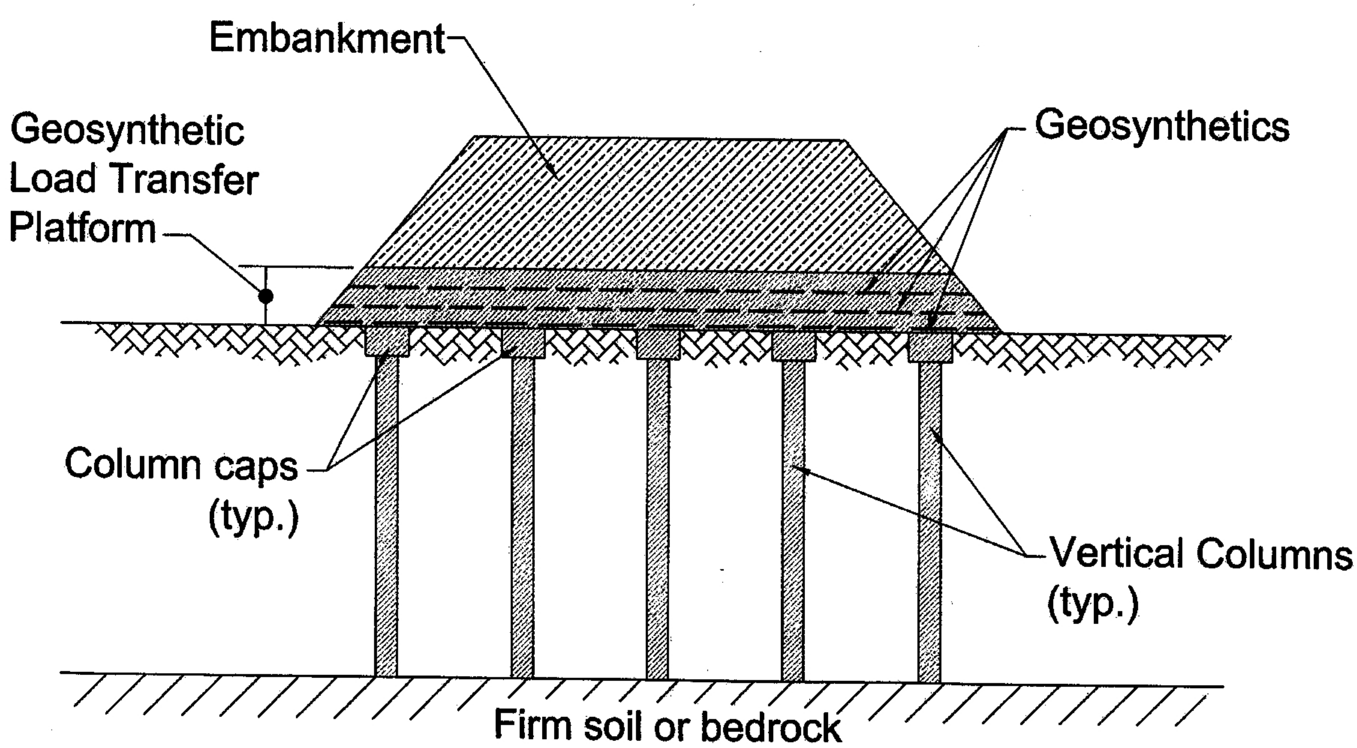

Figure 5. CSE with Column Caps.

Table 1. Possible columns types.

Column Type	Range of Allowable Capacity (kN)	Typical Lengths (m)	Typical Column Diameters (mm)	Typical Installed Cost ($/lm)
Timber pile	100-500	5-20	300-550	25
Steel H pile	400-2,000	5-30	150-300	50-70
Steel pipe pile	800-2,500	10-40	200-1200	
Pre-cast concrete piles	400-1,000	10-15	250-600	50
Cast-in-place concrete shell (mandrel driven)	400-1,400	3-40	200-450	
Shells driven without mandrel	500-1,350	5-25	300-450	
Augered piles	350-700	5-25	300-600	40-60
Micropiles	300-1,000	20-30	150-250	150-300
Deep mix method (DMM)	400-1,200	10-30	600-3000	30-500
Stone columns	100-500	3-10	450-1200	45-60
GEC	300-600	3-10	800-1500	n/a
Geopier rammed aggregate piers	225-650	3-10	600-900	40-90
VCC	200-600	3-10	450-600	80-100
CSV (combined soil stabilization)	30-60	3-10	120-180	n/a
AU-Geo	75-150	2-15	150	n/a

3.4 EQUIPMENT

Column installation typically involves specialized construction equipment. The Technical Summaries on Stone Columns and Soil Mixing provide information on the equipment requirements for stone columns, rammed aggregate piers, VCC, and soil mix columns. The equipment requirements for driven piles may be found in FHWA HI-97-013.

The equipment for most column installation is relatively large and may be heavy. On soft soil projects a working platform may be required to provide access for the equipment. The working platform may include a layer of geosynthetic reinforcement to stabilize the subgrade. This layer of reinforcement is solely for the working platform and should not be included in the LTP analysis.

The equipment used to place and compact the select granular fill used to construct the LTP should be lightweight, low contact pressure equipment in order to minimize the load on the soft soil between columns during fill placement and compaction. Turning of construction equipment on the LTP during construction should be minimized to reduce the potential for displacing or damaging the reinforcement. Figures 6 and 7 show the construction process.

Figure 6. Load Transfer Platform Reinforcement Placement and Compaction.

Figure 7. Load Transfer Platform Select Fill Placement.

CHAPTER 4
DESIGN CONCEPTS

The design of column supported embankments is a complex soil-structure interaction problem. There are currently several empirical methods for the design that focus predominantly on the analysis of the load transfer platform. The methods that will be presented in this chapter include the British Standard (BS8006), the Swedish Standard, the German method, and the Collin method. Each approach has been used successfully on numerous projects, and through those projects, each method has been demonstrated to work well.

4.1 FUNDAMENTAL CONCEPTS

The design of column supported embankments must consider both limit state, and serviceability state failure criteria. The limit state failure modes are shown in Figure 8. The columns must be designed to carry the vertical load from the embankment without failing (Figure 8a). The columns are typically assumed to carry the full load from the embankment. The lateral extent of the columns under the embankment must be determined (Figure 8b). The load transfer platform must be designed to transfer the vertical load from the embankment to the columns (Figure 8c). The potential for lateral sliding of the embankment on top of the columns must be addressed (Figure 8d). Finally, global stability of the system must be evaluated (Figure 8e).

In addition to limit state analysis, serviceability state design must be considered. The strain in the geosynthetic reinforcement used to create the load transfer platform should be kept below some maximum threshold to preclude unacceptable deformation reflection (*i.e.,* differential settlement) at the top of the embankment. Settlement of the columns must also be analyzed to assure that unacceptable settlement of the overall system does not occur, as shown in Figure 9.

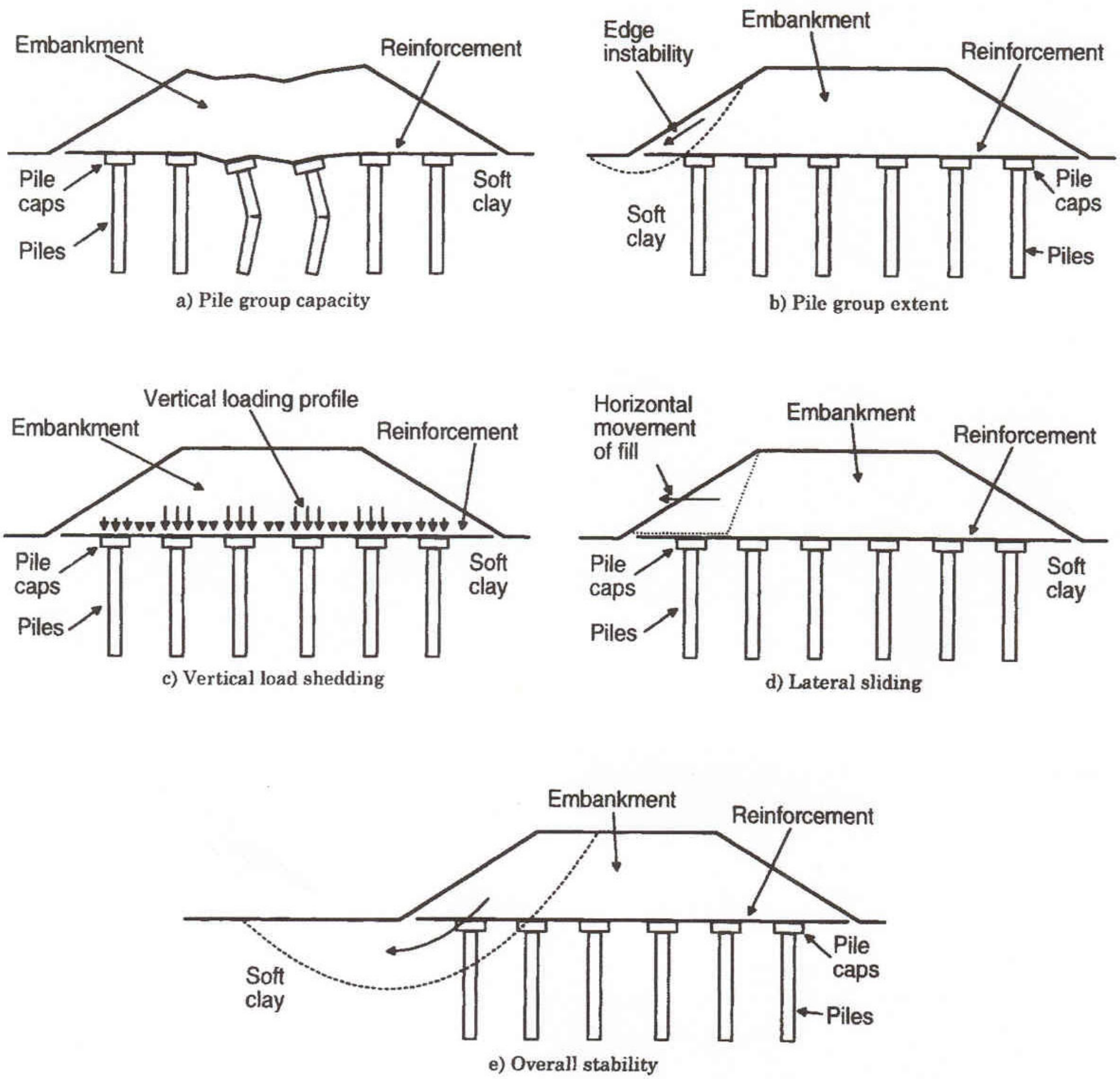

Figure 8. Limit State Failure Modes.(6)

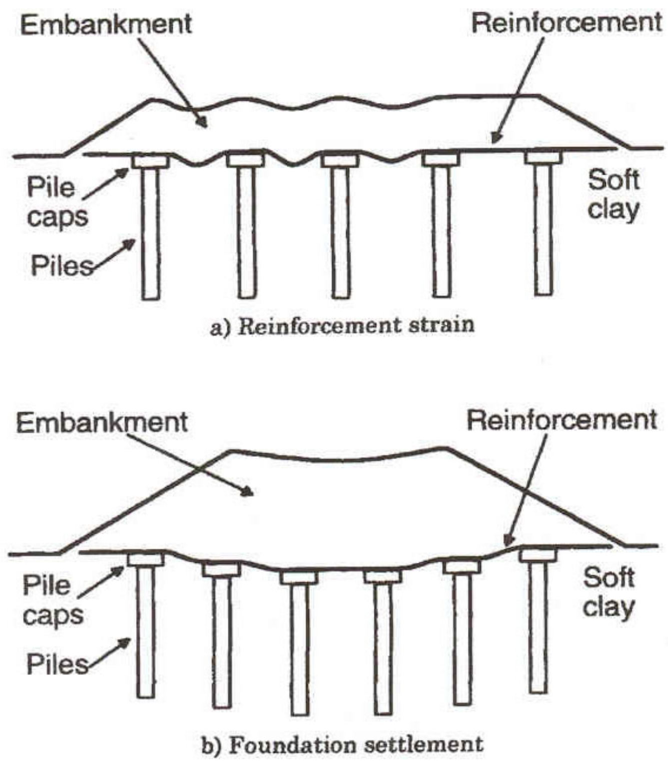

Figure 9. Serviceability State.[6]

The general design steps for a CSE are provided below:

1. Estimate preliminary column spacing (use feasibility assessment guidelines).
2. Determine required column load.
3. Select preliminary column type based on required column load and site geotechnical requirements.
4. Determine capacity of column to satisfy limit and serviceability state design requirements.
5. Determine extent of columns required across the embankment width.
6. Select LTP design approach (*i.e.,* catenary or beam).
7. Determine reinforcement requirements based on estimated column spacing (step 1). Revise column spacing as required.
8. Determine reinforcement requirements for lateral spreading.
9. Determine overall reinforcement requirements based on LTP and lateral spreading.
10. Check global stability.
11. Prepare construction drawings and specifications.

4.2 COLUMN DESIGN

The selection of column type is most often based on constructability, load capacity, and cost. The constructability is discussed in Chapter 3, and cost will be covered in Chapter 6. The load that a column is required to carry is typically based on the tributary area for each column. The embankment and any surcharge load is typically assumed to be carried in their entirety by the columns.

For purposes of determining the design vertical load in the column, it is convenient to associate the tributary area of soil surrounding each column, as illustrated in Figure 10. Although the tributary area forms a regular hexagon about the column, it can be closely approximated as an equivalent circle having the same total area. For square column pattern, the effective diameter (diameter D_e) is equal to 1.13 times the center-to-center column spacing. For triangular column pattern, the effective diameter is equal to 1.05 times the center-to-center column spacing (typical center-to-center column spacing ranges from 1.5-3.0 m (5-10 ft.)).

The required design vertical load (Q_r) in the column is determined according to the following equation:

$$Q_r = \pi(D_e/2)^2 (\gamma H + q) \quad (1)$$

where: D_e = effective tributary area of column
H = height of embankment
q = live and dead load surcharge (typically 12 kN/m^2 (250 psf))
γ = unit weight of the embankment soil

The range of required column loads for a 1.5 m (5 ft.) center-to-center column spacing ranges from approximately 110-250 kN (25-56 kips) for embankment heights ranging from 3-10 m. The required load for a 3 m (10 ft.) center-to-center column spacing is approximately 400-1100 kN (90-250 kips) for embankment heights of 3-10 m (10-33 ft.). After determining the required load in the column, Table 1 (presented in Chapter 3) may be used to select the column type that will provide the required capacity for the lowest costs.

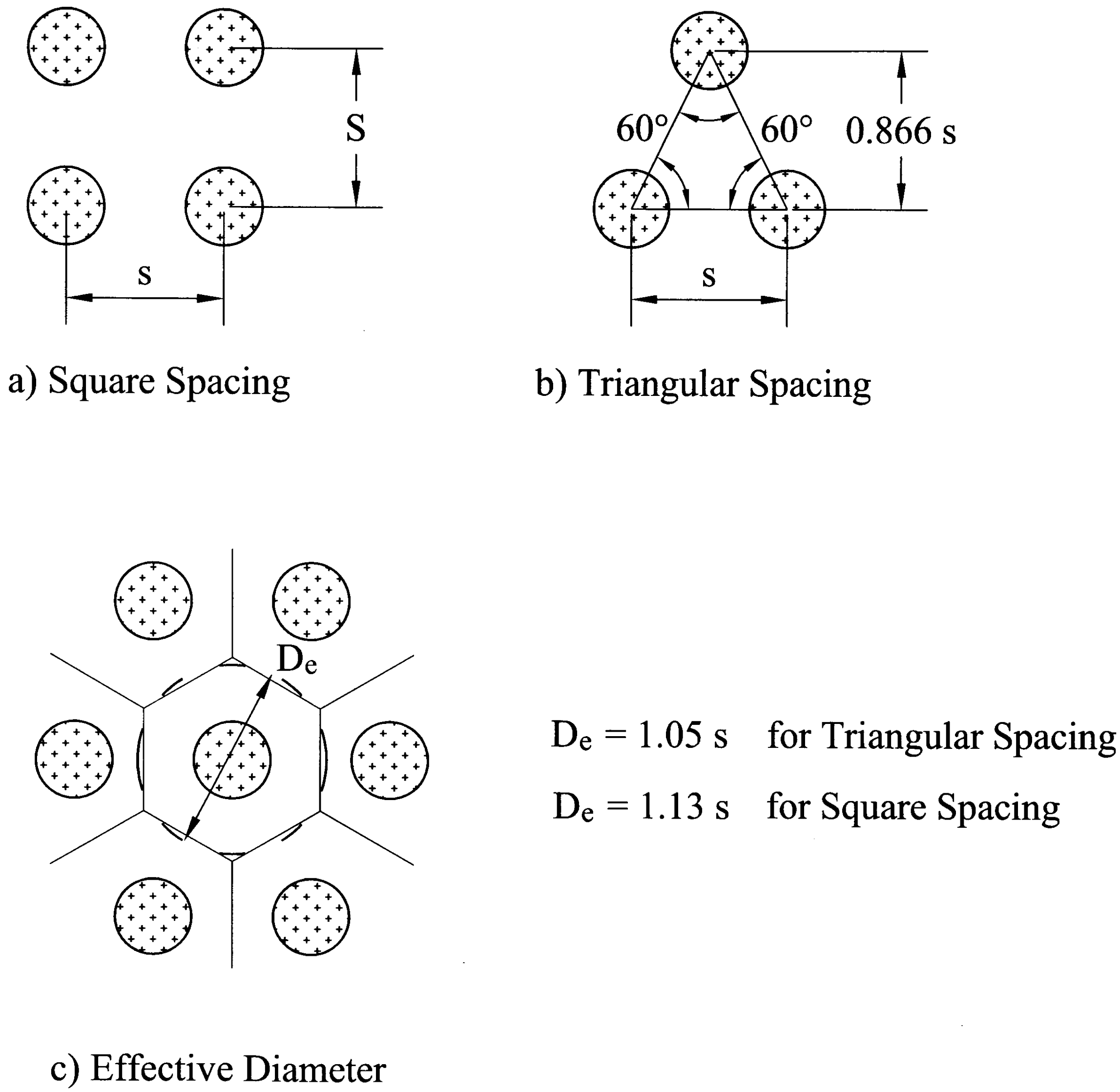

Figure 10. Column Layout.

The design of concrete, steel, and timber piling is well established. Design guidelines have been developed by FHWA and may be found in *Design and Construction of Driven Pile Foundations* (FHWA DTFH61-97-D-00025). For the design of timber piles, the reader is also referred to *Timber Pile Design and Construction Manual,* Timber Piling Council.(7) The design and construction of micropiles is provided in *Micropile Design and Construction Guidelines,* FHWA-SAS-97-070(8).

Soil mix columns, stone columns, rammed aggregate piers, and geotextile encased columns are covered in the other technical summaries of this manual. The vertical load capacity design of VCC, CSV, and Au-Geo is not well defined and is typically performed by the contractor. The design verification for these systems is typically achieved with a static load test. Table 1 provides a listing of potential columns for this application and typical design loads and lengths for each.

4.3 EDGE STABILITY- LATERAL EXTENT OF COLUMNS

The lateral extent of the column system across the width of the embankment should extend a sufficient distance beyond the edge of the embankment to ensure that any instability or differential settlement that occurs outside the column supported area will not affect the embankment crest (Figure 8b). There are several approaches that may be used to check the edge stability. The computer software developed for FHWA for the design of both reinforced and non-reinforced slopes and embankments, ReSSA, is an excellent tool for checking edge stability.

The British Standard (BS8006)**(6)** requires that the columns extend to within a minimum distance (L_p) of the toe of the embankment. Figure 11 defines the terms for edge stability. L_p is determined from the following equation:

$$L_p = H\,(n - \tan\theta_p) \tag{2}$$

where:
- n = side slope of the embankment
- θ_p = is the angle (from vertical) between the outer edge of the outer-most column and the crest of the embankment [$\theta_p = (45 - \varphi_{emb}/2)$].
- φ_{emb} = effective friction angle of embankment fill

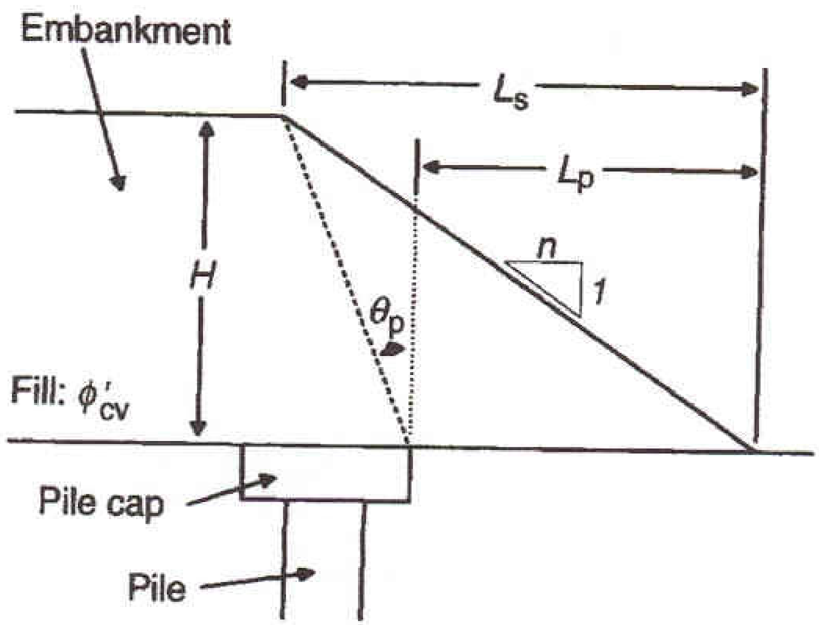

Figure 11. Edge Stability.[6]

The British method is an excellent check of the more rigorous stability analysis using limit equilibrium techniques (*i.e.,* ReSSA). For preliminary designs and/or feasibility analysis, the simplified British approach is sufficient.

4.4 LATERAL SPREADING

The potential for lateral spreading of the embankment must be analyzed (Figure 12). The geosynthetic reinforcement must be designed to prevent lateral spreading of the embankment. This is a critical aspect of the design, as many of the columns that are appropriate for column supported embankments are not capable of providing adequate lateral resistance to prevent spreading of the embankment without failing.

The geosynthetic reinforcement must be designed to resist the horizontal force due to the lateral spreading of the embankment. The required tensile force to prevent lateral spreading (T_{ls}) is determined from the following equation:

$$T_{ls} = K_a (\gamma H + q)H/2 \quad (3)$$

where:

K_a = coefficient of active earth pressure ($\tan^2 (45-\varphi_{emb}/2)$)

The minimum length of reinforcement (L_e) necessary to develop the required strength of the reinforcement without the side slope of the embankment sliding across the reinforcement is determined using the equation below:

$$L_e = T_{ls} /[0.5 \gamma H(c_{iemb} \tan \varphi_{emb})] \quad (4)$$

where:

c_{iemb} = coefficient of interaction for sliding between the geosynthetic reinforcement and embankment fill

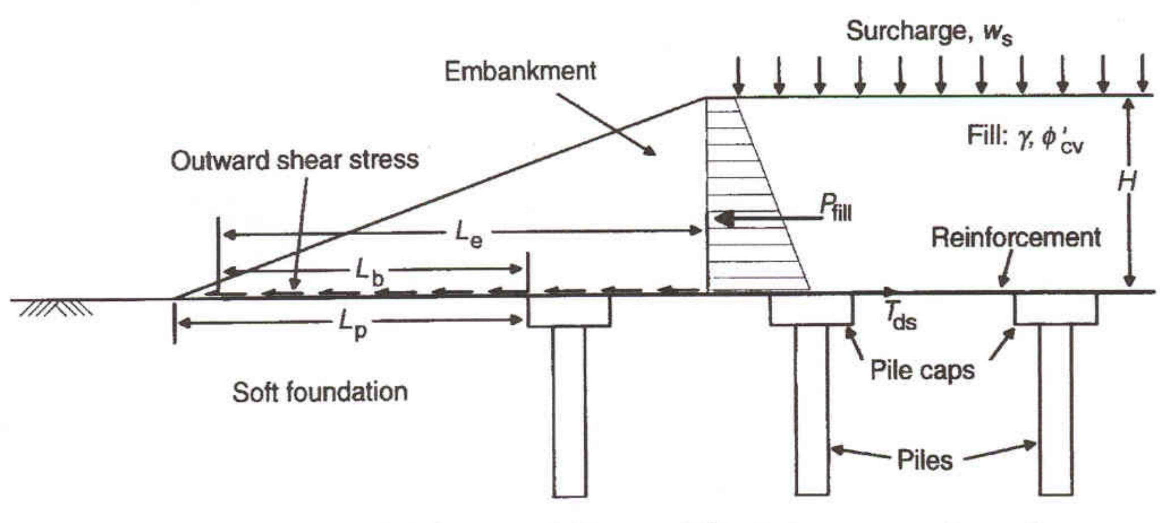

Figure 12. Lateral Spreading.(6)

4.5 LOAD TRANSFER PLATFORM DESIGN

There are two fundamentally different approaches to the design of the load transfer platform. The first approach, which is used by the British Standard(6), the Swedish(9)(10), and the German methods(11)(12), is for the reinforcement to act as a catenary. The reinforcement transfers the load from the embankment fill to the columns through catenary tension in the reinforcement, as shown in Figure 13. In essence, the reinforcement behaves as a structural element, and any benefits achieved by the creation of a composite reinforced soil mass are ignored. The primary assumptions in the catenary theory are:

- Soil arch forms in the embankment.
- Reinforcement is deformed during loading.
- One layer of reinforcement is used; if more than one layer of reinforcement is used, only the tensile strength of the multiple layers is considered.

The second approach for the design of the load transfer platform (Collin Method) is to use multiple layers of reinforcement to create a stiff reinforced soil mass. The Collin Method is a refinement of a method sometimes referred to as the Guido Method(13)(14)(15). The reinforced soil mass acts as a beam to transfer the load from the embankment above the platform to the columns below. The primary assumptions for the beam theory are:

- A minimum of three layers of reinforcement is used to create the platform.
- Spacing between layers of reinforcement is 200-450 mm (8-18 in.).
- Platform thickness is greater than or equal to one-half the clear span between columns.
- Soil arch is fully developed within the depth of the platform.

The catenary method generally requires higher strength reinforcement for the same design conditions, as opposed to the beam method (*i.e.,* column spacing and embankment height). The beam method will generally allow for larger column-to-column spacing than the catenary method for standard geosythetics (*i.e.,* materials available off the shelf).

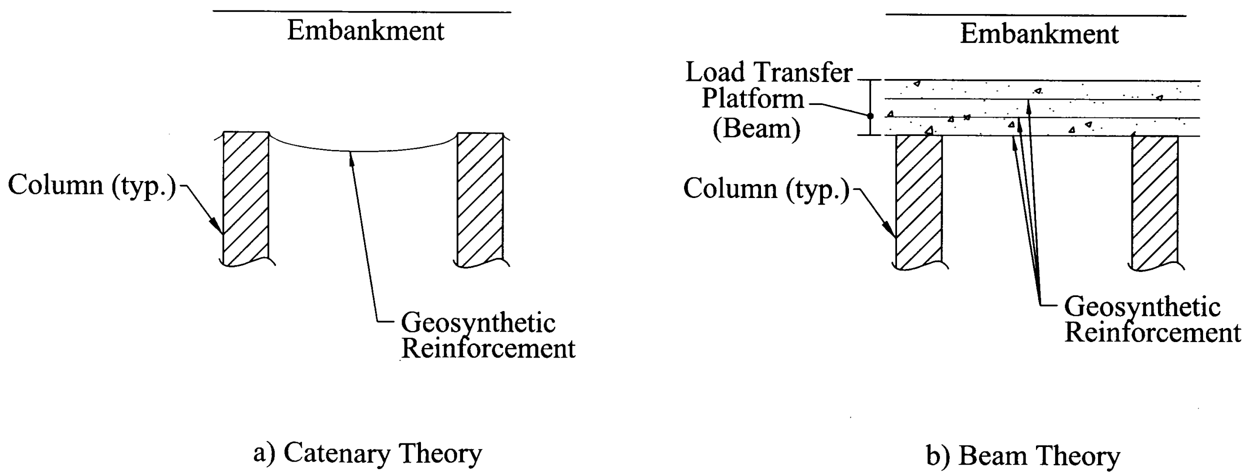

Figure 13. Load Transfer Mechanisms.

a. Soil Arching

All load transfer platform design methods covered in this Technical Summary consider soil arching. Soil arching is defined by McNulty[16] as "the ability of material to transfer loads from one location to another in response to a relative displacement between locations." Figure 14 demonstrates the concept of soil arching. The stress at point "a" in Figure 14a is equal to the overburden stress γH, where γ is the unit weight of the soil, and H is the height of the soil mass. At the moment when the soil loses support, a temporary true arch is formed. The soil at point "a" is in tension, and the weight of the soil prism starts to be transferred to the adjacent unyielding soil (Figure 14b). Deformation within the temporary true soil arch occurs. As the soil settles into an inverted arch (Figure 14c), an equilibrium state is achieved, the adjacent unyielding soil mobilizes its shear strength, and the load transfer is complete. At some height (H_e) above point "a," the transfer of stress is complete. The settlements in the soil mass above this point are uniform.[17]

The degree of soil arching is defined as the soil arch ratio (ρ), which is the ratio of the average vertical stress on the yielding portion (*i.e.,* soft soil between columns) to the average vertical stress due to the embankment fill and surcharge load.

$$\rho = \sigma_s/(\gamma H+q) \tag{5}$$

where: σ_s = average vertical stress applied between columns (*i.e.,* trapdoor)

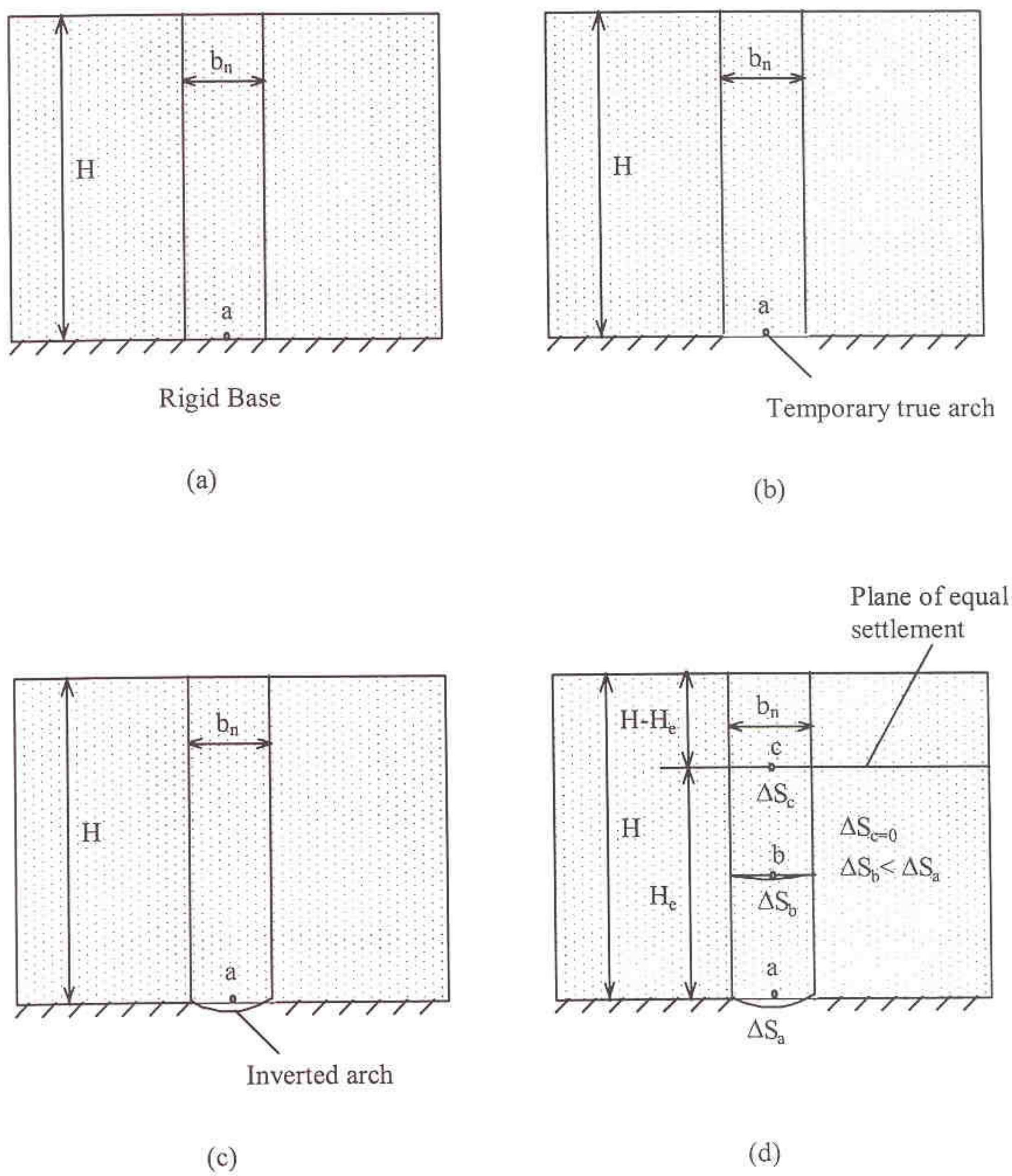

Figure 14. Soil Arching.[(17)]

b. Tension Membrane

In addition to soil arching, the load transfer platform design includes tension membrane theory. The vertical load from the soil within the arch and any surcharge load, if the thickness of the embankment is not great enough to develop the full arch, is carried by the reinforcement. There are several theories available to estimate the tension in the reinforcement (Fluet[18] and Giroud[19]). A detailed discussion on tension membrane theory is beyond the scope of this Technical Summary.

The symbols used by the British Standard, the Swedish, the German, and the Collin Methods have been standardized for ease of reference. Figure 15 shows the common symbols that will be used in presenting these methods. They are defined below:

d	=	diameter of the column
H	=	height of embankment
P_c'	=	vertical stress on the column
q	=	surcharge load
s	=	center-to-center column spacing
T_{RP}	=	tension in the extensible reinforcement
W_T	=	vertical load carried by the reinforcement

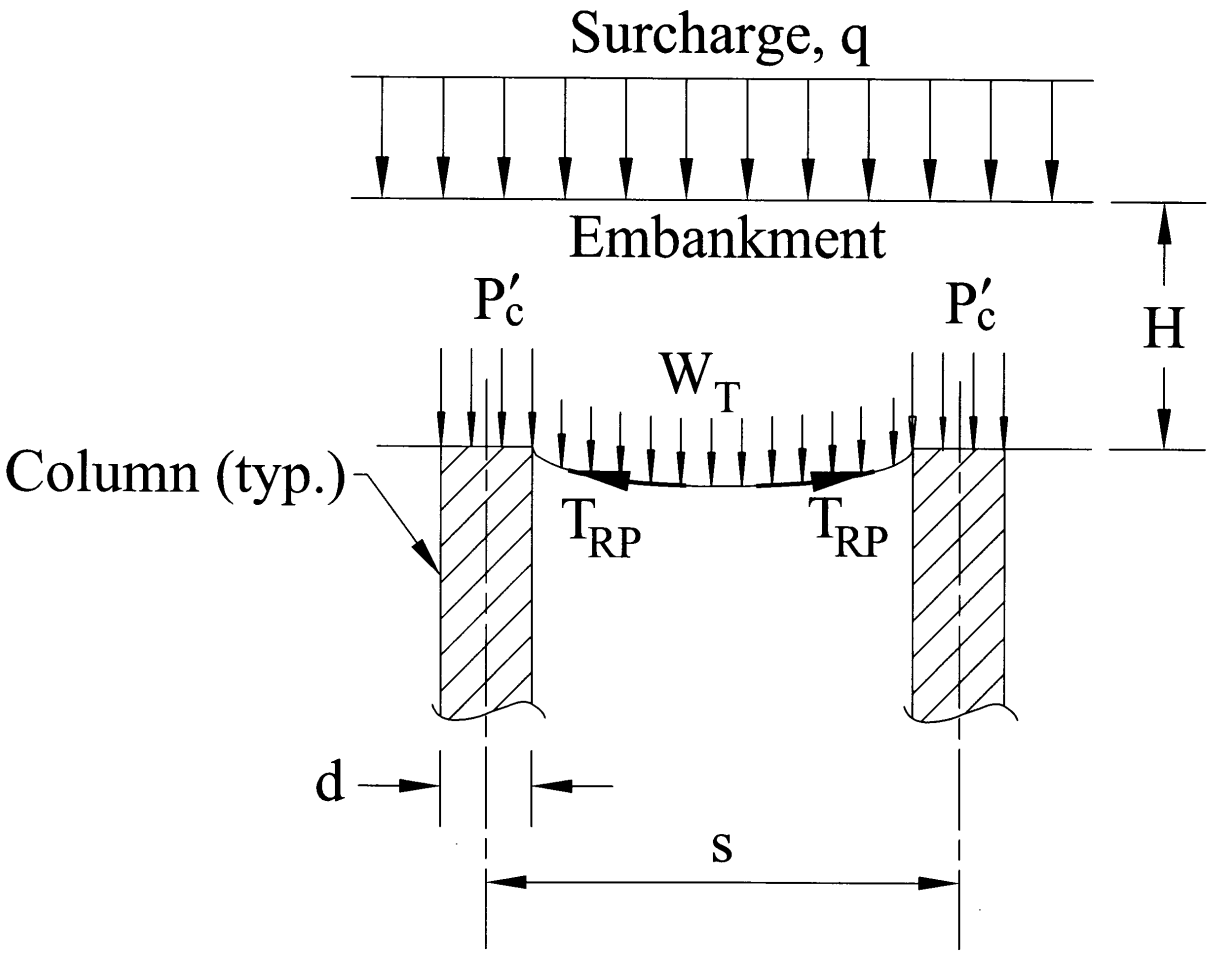

Figure 15. Definition of Terms.

c. British Standard BS 8006 [6]

The British Standard recommends that the embankment height be a minimum of 1.4 times the clear span between columns. This is to ensure that differential settlement cannot occur at the surface of the embankment. Soil arching between adjacent columns induces greater vertical stresses on the columns than on the surrounding foundation soil. The ratio of the vertical stress on the columns to the average vertical stress at the base of the embankment is determined from the following equation and is based on Marston's formula[20]:

$$P'_c/\sigma'_v = [C_c d/H]^2 \tag{6}$$

where: P'_c = vertical stress on the column

σ'_v = the average vertical stress at the base of the embankment

= $(f_{fs}\ \gamma H + f_q\ q)$
f_{fs} = partial soil unit mass load factor (1.3)
f_q = partial surcharge load factor (1.3)
C_c = arching coefficient
= (1.95H/d – 0.18) for end bearing columns (unyielding)
= (1.50H/d – 0.07) for frictional columns (normal)
d = column diameter

The vertical load carried by the reinforcement spanning between columns for the case where H > 1.4 (s-d) may be determined as follows:

$$W_T = [1.4\ s\ f_{fs}\ \gamma\ (s\text{-}d)/(s^2\text{-}d^2)](s^2\text{-}d^2(P'_c/\sigma'_v) \qquad (7)$$

where: s = center-to-center spacing between columns

For the case where: $0.7(s\text{-}d) \leq H \leq 1.4\ (s\text{-}d)$, the distributed vertical load carried by the reinforcement is determined from the following equation:

$$W_T = [(s\ f_{fs}\ \gamma\ H + f_q\ q)/(s^2\text{-}d^2)](s^2\text{-}d^2(P'_c/\sigma'_v) \qquad (8)$$

The tension in the extensible reinforcement (T_{rp}) per lineal meter of reinforcement resulting from the distributed load is

$$T_{rp} = 0.5\ W_T\ [(s\text{-}d)/d](1+1/6\varepsilon)^{0.5} \qquad (9)$$

where: ε = strain in the reinforcement

The initial tensile strain in the reinforcement is needed to generate a tensile load. A practical upper limit of 6 percent strain should be imposed to ensure all embankment loads are transferred to the piles.

The tensile load (T_{rp}) develops as the reinforcement deforms under the weight of the embankment. This normally occurs during construction of the embankment, but in-situations where the reinforcement cannot deform during construction, the reinforcement will not carry the applied loads until the foundation settles. The above equation is appropriate for those reinforcements that can undergo deformation during loading (*i.e.,* extensible reinforcements). For inextensible reinforcements, alternative relationships should be used to determine their required strength.

The long-term strain in the reinforcement (due to creep) should be kept to a minimum to ensure that the long-term localized deformations do not occur at the surface of the embankment. A minimum creep strain of 2 percent over the design life of the reinforcement should be allowed.

d. Swedish Method[9]

The Swedish method has many similarities to the British Standard. The Swedish method is valid when the following assumptions/parameters are satisfied:

- Arch formation occurs
- Reinforcement is deformed during loading
- One (1) layer of reinforcement is used
- Reinforcement is located within 0.1 m (4 in.) above the column
- Embankment height is greater than or equal to the clear distance between columns
- Ratio of column or column cap area to influence area per column is greater than or equal to 10 percent
- Embankment fill effective friction angle is 35°
- Initial strain in the reinforcement is limited to 6 percent
- Long-term (creep) strain is limited to 2 percent
- Total strain is less than 70 percent strain at failure

Figure 16 shows the model used in the Swedish method to determine vertical load carried by the reinforcement. The cross-sectional area of the soil under the arch, which is the load carried by the reinforcement, is approximated using the soil wedge shown in Figure 16. This applies even when the embankment height is lower than the top of the soil wedge (*i.e.,* (s-d)/(2tan 15°)).

The two-dimensional weight (W_T) of the soil wedge is determined from the following equation:

$$W_T = (s-d)^2\gamma/(4 \tan 15°) \text{ per unit length in depth} \tag{10}$$

The three-dimensional effects are estimated through load distribution, where the load is distributed over the surface according to Figure 17 and is taken up by the reinforcement along the edge of the column. The force in the reinforcement, per lineal meter of depth, due to the vertical load in three-dimensions is calculated using the equation below:

$$T_{rp} = 0.5\ [1+(s/d)]\ W_T\ (1+1/6\varepsilon)^{0.5} \tag{11}$$

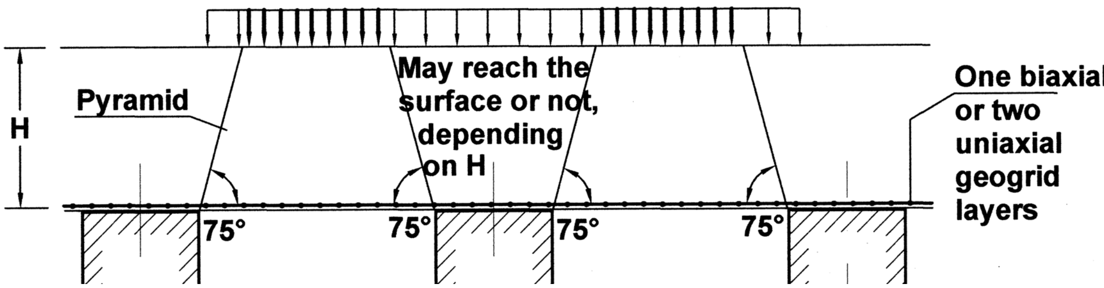

Figure 16. Swedish Method Soil Arch.[9]

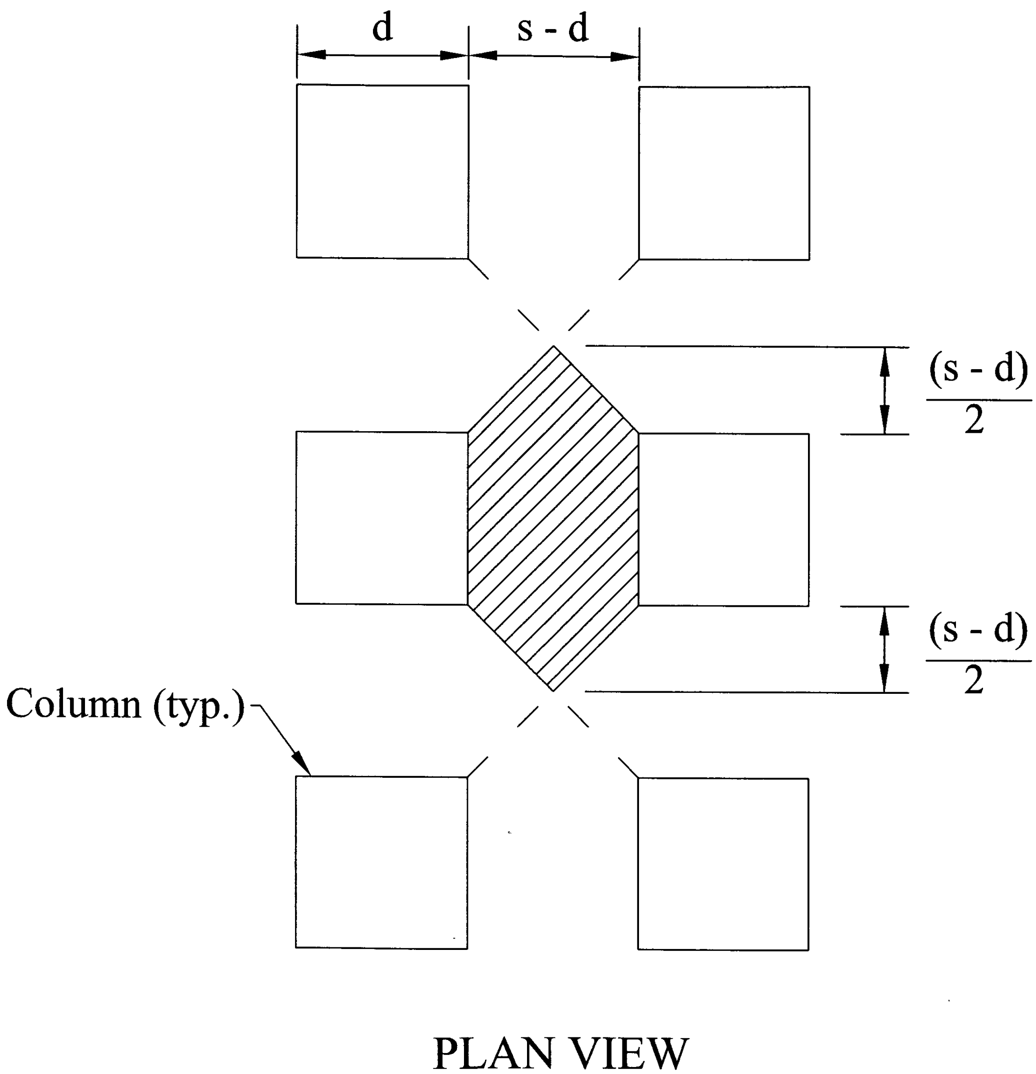

Figure 17. Swedish Method Load Distribution between Columns.[9]

e. German Method[11][12]

The German method, unlike either the British Standard or the Swedish method, considers the effect of the soft foundation soil in determining the load carried by the reinforcement. Specifically, the undrained shear strength of the foundation soil is considered to provide some resistance to the vertical load from the embankment. The German method also directly considers the shear strength of the embankment material in determining the arching within the embankment. This method is only valid when the height of the embankment (H) is greater than the column spacing(s).

Two failure criteria are considered: failure of the embankment fill at the crown of the arch (typically controls for light surcharge loads and large column spacing); and failure at the bearing point of the arch. The ratio (E) of the vertical load on the columns to the average load at subgrade is a function of which failure mode controls the design.

For failure to occur at the crown of the arch, E is determined from the equation below. This condition occurs for relatively shallow embankments with wide column spacing.

$$E = 1 - [1-(d/s)^2](A-AB+C) \tag{12}$$

where:

$$A = [1-(d/s)]^{2(Kp-1)} \tag{13}$$

$$B = [s/(1.41H)][(2K_p - 2)/(2K_p - 3)] \tag{14}$$

$$C = [(s-d)/(1.41H)]\,[(2K_p - 2)/(2K_p - 3)] \tag{15}$$

$$K_p = (1+ \sin \varphi')/(1- \sin \varphi') \tag{16}$$

φ' = effective friction angle of the embankment fill

Failure at the bottom of the arch must also be analyzed with the following equation:

$$E = \beta/(1+\beta) \tag{17}$$

where:

$$\beta = [2\,K_p/(K_p + 1)(1 + d/s)][(1-d/s)^{-Kp} - (1 + K_p d/s)] \tag{18}$$

The minimum value of E controls the stress applied to the soil between columns (σ_s). The stress that is applied to the soil between columns (Figure 18) is determined as follows:

$$\sigma_s = [(\gamma H + q)/(s^2 - d^2)][1 - E]s^2 \tag{19}$$

The geosynthetic reinforcement is subjected to the stress on the soil (σ_s) less a vertical reaction stress produced by the supporting effect of the soil between columns (*i.e.,* the bearing capacity). The factored (*e.g.,* safety factor of 2) undrained shear strength (c_u) of the soil is used to determine the bearing capacity of the foundation soil. The equation for determining this allowable stress (σ_o) is shown below:

$$\sigma_o = [(2 + \pi)\, c_u]/FS \tag{20}$$

where: FS = factor of safety for undrained shear strength (typically 2)

The vertical load (W_T) on the geosynthethic reinforcement spanning between columns is determined as follows:

$$W_T = [\sigma_s\, (s^2 - d^2)/2(s' - d)] - [\sigma_o\, ((s^2 - d^2)/2(s' - d)] \text{ per lineal meter} \tag{21}$$

where: s' = s for square column pattern
= 1.4s for triangular column pattern

For the case of more than one layer of reinforcement, the vertical load (W_T) may be distributed between the layers proportionally to their strain resistance. The German method, presented above, is based on the reinforcement being located vertically, less than 0.5 m (1.6 ft.) above subgrade. Special procedures are provided by the German method for the case where the reinforcement is located between 0.5 m and 1 m (1.6 ft. and 3.3 ft.) above subgrade[(21)].

The tensile force in the reinforcement per unit length of reinforcement is determined based on catenary tension and is determined by the following equation:

$$T_{rp} = W_T\, [(s' - d)/2d]\, (1 + 1/6\varepsilon)^{0.5} \tag{22}$$

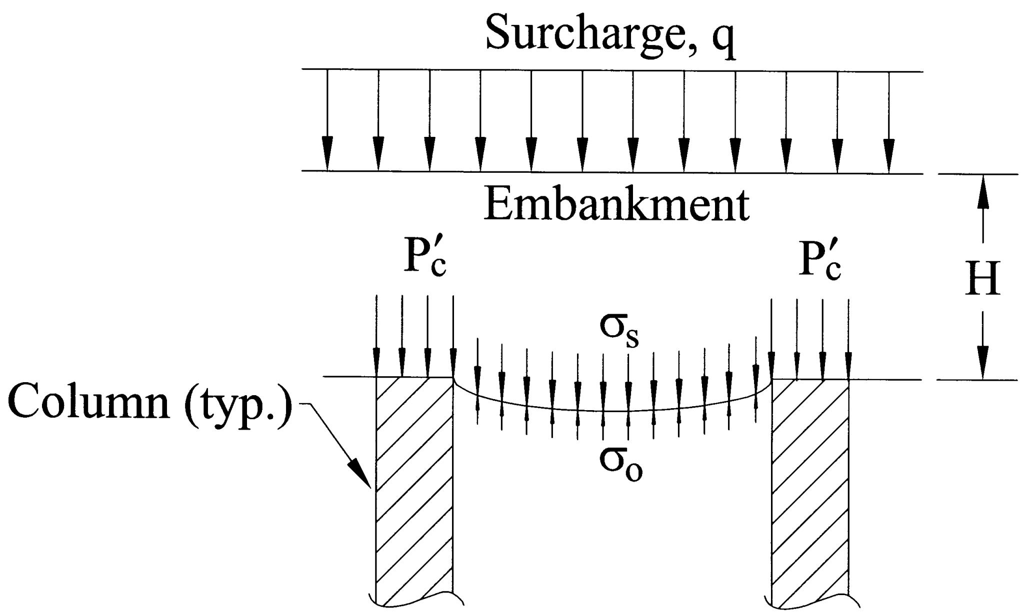

Figure 18. German Method.

f. Collin Method

The Collin Method is fundamentally different than the British Standard, the Swedish Method, or the German Method. The Collin Method is based on the premise that the reinforcement (minimum of three layers of reinforcement) creates a stiffened beam of reinforced soil that distributes the load from the embankment above the load transfer platform (*i.e.,* stiffened beam) to the columns below the platform (Figure 19).

The Collin Method is based on the following assumptions:

- The thickness (h) of the load transfer platform is equal to or greater than one-half the clear span between columns (½ (s-d)).
- A minimum of three layers of extensible (geosynthetic) reinforcement is used to create the load transfer platform.
- Minimum distance between layers of reinforcement is 200 mm (8 in.).
- Select fill is used in the load transfer platform.
- The primary function of the reinforcement is to provide lateral confinement of the select fill to facilitate soil arching within the height (thickness) of the load transfer platform.
- The secondary function of the reinforcement is to support the wedge of soil below the arch.

- All of the vertical load from the embankment above the load transfer platform is transferred to the columns below the platform.
- The initial strain in the reinforcement is limited to 5%.

The vertical load carried by each layer of reinforcement is a function of the column spacing pattern (*i.e.,* square or triangular) and the vertical spacing of the reinforcement. If the subgrade soil is strong enough to support the first lift of fill, the first layer of reinforcement is located 0.15-0.25 m (6-10 in.) above subgrade. Each layer of reinforcement is designed to carry the load from the platform fill that is within the soil wedge below the arch. The fill load attributed to each layer of reinforcement is the material located between that layer of reinforcement and the next layer above (Figure 20).

The uniform vertical load on any layer (n) of reinforcement (W_{Tn}) may be determined from the equation below:

$$W_{Tn} = \text{(area at reinforcement layer n + area at reinforcement layer (n+1))/2)}$$
$$\text{(layer thickness) (load transfer platform fill density)/(area at reinforcement layer n)}$$

$$W_{Tn} = [(s\text{-}d)_n^2 + (s\text{-}d)_{n+1}^2]\sin 60°\, h_n\, \gamma / [(s\text{-}d)_n^2 \sin 60°] \quad \text{for triangular pattern} \quad (23)$$

$$W_{Tn} = [(s\text{-}d)_n^2 + (s\text{-}d)_{n+1}^2]\, h_n\, \gamma / (s\text{-}d)_n^2 \quad \text{for square pattern} \quad (24)$$

The tensile load in the reinforcement is determined based on tension membrane theory [(19)] and is a function of the amount of strain in the reinforcement. The tension in the reinforcement is determined from the following equation:

$$T_{rpn} = W_{Tn}\, \Omega\, D/2 \quad (25)$$

where: D = design spanning for tension membrane
= $(s\text{-}d)_n$ for square column spacing
= $(s\text{-}d)_n \tan 30°$ for triangular column spacing
= dimensionless factor

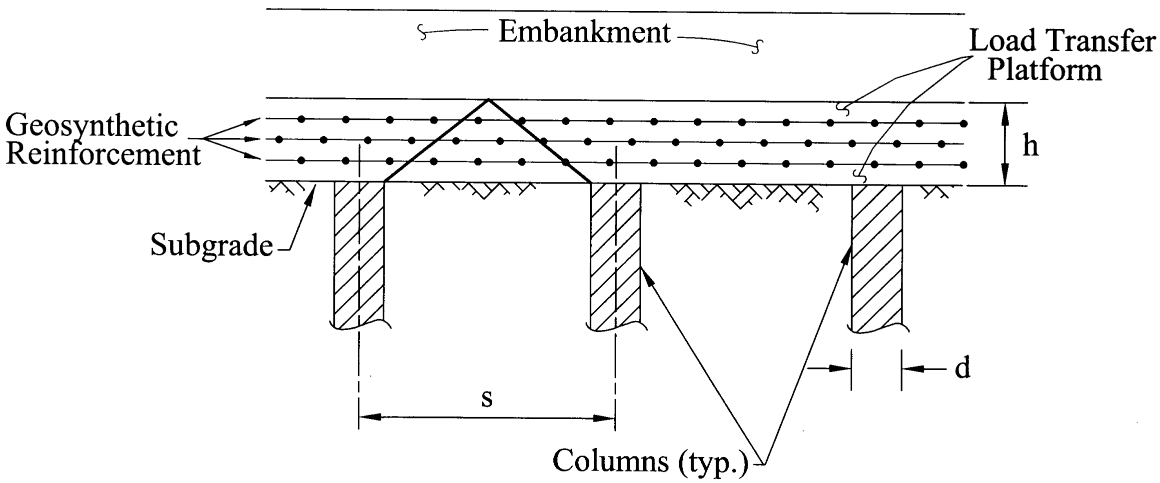

Figure 19. Load Transfer Platform.

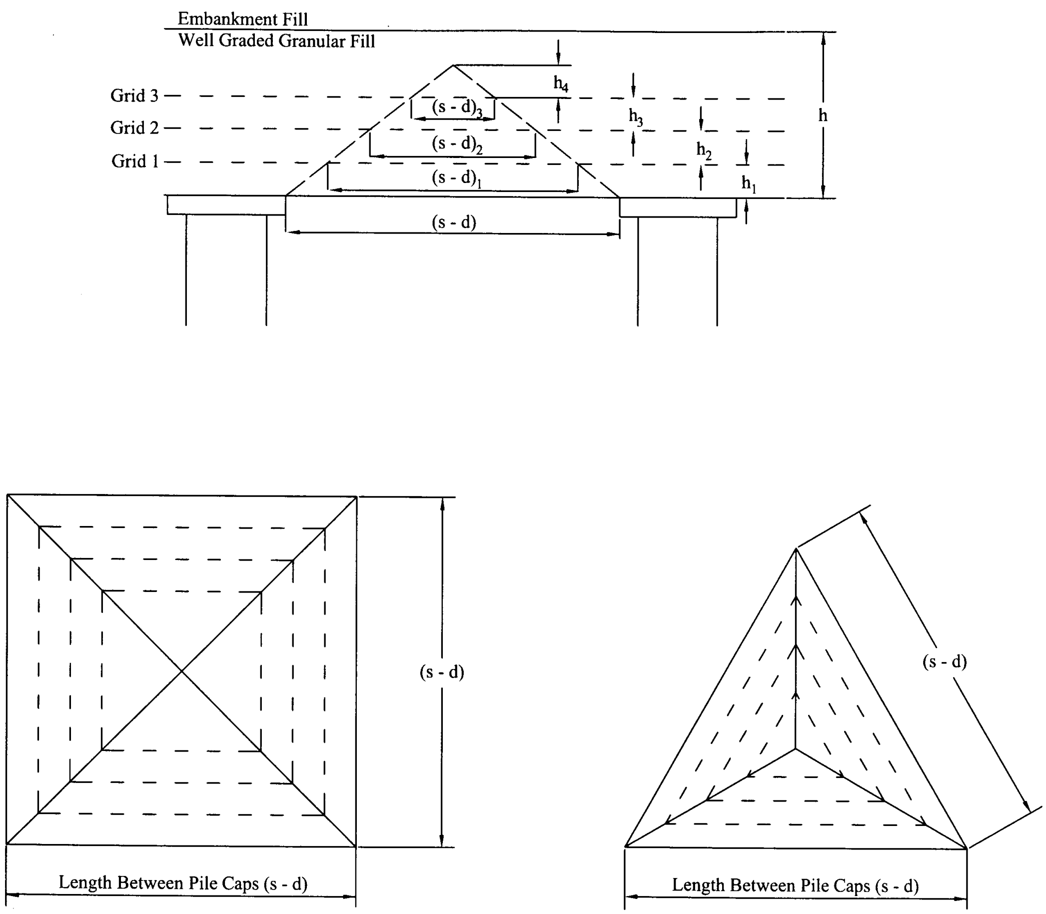

Figure 20. Load Transfer Platform Design Collin Method.

Table 2. Values of Ω.

Ω	Reinforcement Strain (ε)%
2.07	1
1.47	2
1.23	3
1.08	4
0.97	5

g. Modified Collin (Beam) Method

Based on research recently completed (Collin, et. al., 2005) using numerical modeling the above procedure has been modified. The modification involves the addition of one layer of reinforcement at subgrade. This layer of reinforcement is designed as a catenary to carry the load from the soil below the arch (Figure 21).

The uniform vertical load on the catenary layer of reinforcement (W_{TC}) may be determined from the equation below:

W_{TCn} = (volume pyramid below the arch) (load transfer platform fill density)/(area at reinforcement catenary layer)

$$W_{Tn} = h_n \gamma / 3 \qquad \text{Square or Triangular column spacing} \qquad (26)$$

The tensile load in the reinforcement is determined based on tension membrane theory and is a function of the amount of strain in the reinforcement. The tension in the reinforcement is determined from the following equation:

$$T_{rpC} = W_{TC}\,\Omega\,D/2 \qquad (27)$$

where: D = design span for tensioned membrane
= 1.41*[(s-d) - 2(∑Vertical Spacing/tan45)] for square column spacing
= 0.867*[(s-d) - 2(∑Vertical Spacing/tan45)] for triangular column spacing
Ω = dimensionless factor from tensioned membrane theory

The reinforcement to create the beam above the catenary layer of reinforcement is designed according to equations 23-25.

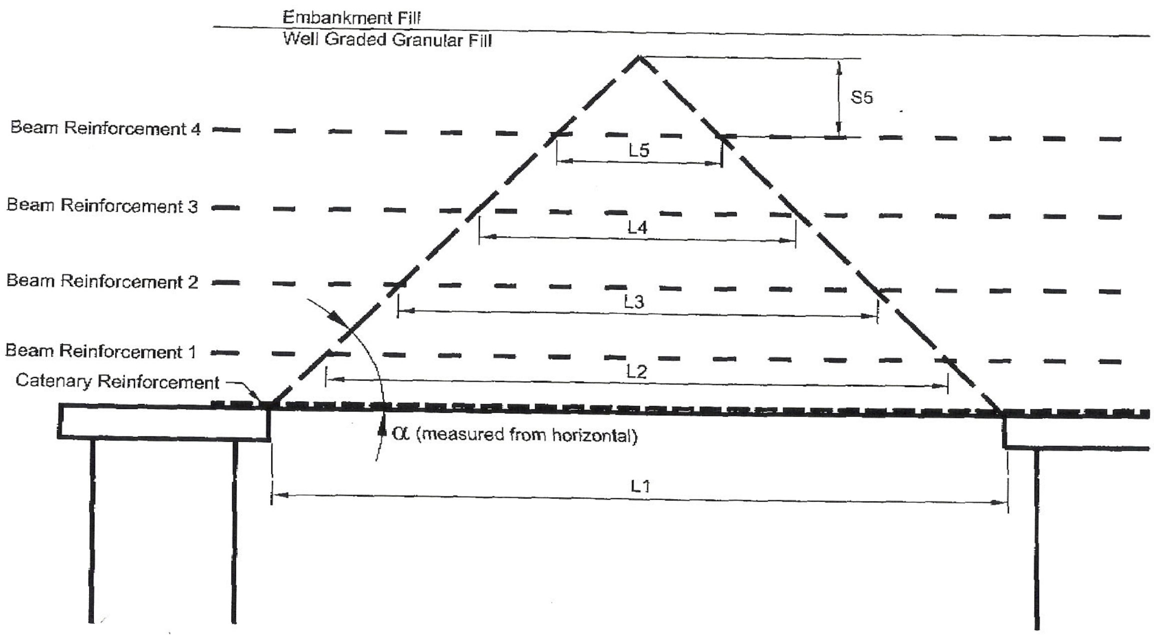

Figure 21. Modified Collin Method Reinforcement

h. Summary of Methods

The design methods summarized in this section have many similarities. However, there are significant differences between the approaches that deal with the fundamental concepts of the load transfer platform. Table 3 provides a summary of these differences.

Table 3. Load transfer platform design method summary.

Method	Design Approach	Number of Reinforcement Layers	Angle of Arch from Horizontal	Subgrade Support	Allowable Strain (ε) in Reinforcement (%)
British	catenary	1*	70	No	6
Swedish	catenary	1*	75	No	6
German	catenary	1*	n/a	Yes	5-6
Collin	beam	≥ 3	45	Yes	5

** More than one layer of reinforcement may be used, based on the required strength of the reinforcement. However, the layers are not discretely placed with a minimum of 200 mm (8 iin.) between layers, as in the Collin Method.*

4.6 REINFORCEMENT TOTAL DESIGN LOAD

Independent of the method used to analyze the LTP, the total (maximum) design load (T_{total}) in the geosynthetic reinforcement should be determined as follows:

$T_{total} = T_{rp}$ in the direction along the length of the embankment

$T_{total} = T_{rp} + T_{ls}$ in the direction across the width of the embankment

4.7 GLOBAL STABILITY

Global stability of column supported embankments may be evaluated using limit equilibrium computer software, taking into consideration the added shear resistance of the columns and the tensile capacity of the geosynthetic reinforcement. Figure 22 shows the approach used in the British Standard for incorporating the benefit of the columns and geosynthetic. For more guidance on incorporating the benefit of the columns into the global stability analysis see the Technical Summary on Stone Columns. For guidance on incorporating the benefit of geosynthetic reinforcement in the overall stability of the CSE, see *Mechanically Stabilized Earth Walls and Reinforced Soil Slopes* (FHWANHI-00-043).(22)

While it is recommended that the global stability of the CSE be evaluated, it is the author's opinion that if the CSE behaves as a true column supported embankment, there is very little potential for a global stability problem.

4.8 SETTLEMENT

Total settlement of the CSE will be a function of the column design. For methods to estimate settlement, the reader is referred to the previously listed references for each column type. Differential settlement between columns caused by the LTP should be less than 20-30 mm (0.8-1.2 in.) when the LTP design follows the guidelines established for the British, Swedish, or Collin method. When using the German method, differential settlement calculations are required.

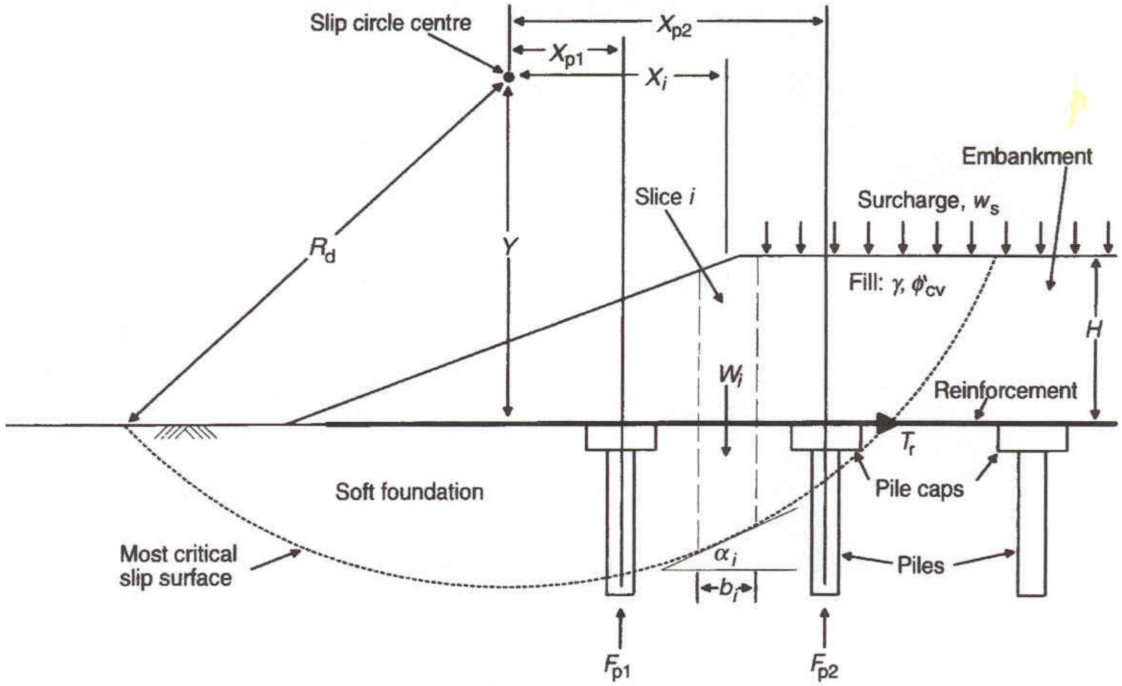

Figure 22. Variables Used in Global Stability Analysis.[6]

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 column supported embankments, 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 technology, a method specification may be utilized.

5.1 METHOD SPECIFICATION

The following method specification is provided as a guide for the installation of the load transfer platform. Details on specifications for the various column types (*i.e.,* stone columns, deep soil mixing, etc.) may be found in the other Technical Summaries. This specification should be modified as appropriate for the particular requirements of the project.

Guide Specification
Column Supported Embankment
Load Transfer Platform

1.0 Materials

1.1 Select Reinforced Fill – reinforced fill materials shall meet the following gradation requirements:

Size	% Passing
200 mm (4 in.)	100
38 mm (1½ in.)	95-100
4.75 mm (No. 4)	65-40
0.425 mm (No. 40)	40-20
0.075 mm (No. 200)	0-15

Reinforced fill material passing the No. 40 sieve shall have a liquid limit less than 40 and a plasticity index less than 20. Reinforced fill materials shall be classified per the unified soil classification system as GW or GW-GM.

1.2 Geosynthetic reinforcement shall have the following properties:

Creep limited strength at 5% strain _____

Ultimate Strength per ASTM D 6637 of _____

Coefficient of Interaction for direct sliding per ASTM 5321 of _____

2.0 Technical Requirements

2.1 Prior to construction of the load transfer platform, the Contractor shall prepare subgrade, and remove any deleterious materials. The foundation soil shall be observed and approved by the on-site Geotechnical Engineer prior to placement of select reinforced fill.

2.2 Select reinforced fill shall be placed in horizontal layers not exceeding 250 mm (10 in.) in uncompacted thickness for heavy compaction equipment. For zones where compaction is accomplished with hand-operated compaction equipment, fill shall be placed in horizontal layers not exceeding 150 mm (6 in.) in uncompacted thickness.

2.3 Select reinforced fill shall be compacted to a minimum 95% maximum dry density, as determined in accordance with ASTM D-1557 (modified proctor) at a moisture content no greater than 2 percent above or below optimum.

2.4 Test methods and frequency, and verification of material specifications and compaction, shall be the responsibility of the State.

2.5 An approved set of construction drawings and contract specifications shall be on-site at all times during construction of the load transfer platform.

3.0 Geosynthetic Reinforcement Placement

3.1 The reinforcement shall be placed at the locations and elevations shown on the drawings.

3.2 Construction equipment shall not be operated directly on the geosynthetic reinforcement. A minimum fill thickness of 150 mm (6 in.) is required for operation of vehicles over the reinforcement. Turning of vehicles should be kept to a minimum to prevent tracks or tires from displacing the fill and/or geosynthetic reinforcement.

3.3 Minimum overlap of adjacent rolls of reinforcement shall be per the construction drawings.

4.0 Changes to Reinforcement Layout or Placement

4.1 No changes to the geosynthetic reinforcement layout, including, but not limited to length, reinforcement type (*i.e.,* strength), direction of reinforcement, or elevation shall be made without the explicit written approval of the Engineer.

5.0 Measurement and Payment

5.1 The measurement for payment of the load transfer platform shall be based on the plan area of the platform.

5.2 The Contractor shall be paid a unit price per square meter of plan area for supply and installation of the load transfer platform, including the select fill and the geosynthetic reinforcement.

5.2 PERFORMANCE SPECIFICATION

Performance specifications shall include the design and installation of the columns, as well as the load transfer platforms. Specifications for the various column types are beyond the scope of this document. The reader is referred to *Design and Construction of Driven Pile Foundations* (FHWA DTFH61-97-D-00025) and *Micropile Design and Construction Guidelines* (FHWA-SAS-97-070) for more information on performance specifications for piles. For soil mix columns, stone columns, rammed aggregate piers, and geotextile encased columns, see the other Technical Summaries of this manual.

This section will deal solely with the design and construction of the load transfer platform. The specification shall clearly define the modes of failure that must be analyzed as part of the design/build Contractor's submittal. However, the choice of design methods should be left to the Contractor. If the specifying agency wants a specific design approach used for the design of the load transfer platform, then a method specification should be used.

Documentation

The Contractor shall furnish shop drawings to the Engineer for review 2 weeks prior to the start of work, indicating the thickness of the load transfer platform, the spacing and pattern of the columns, the number of reinforcement layers and vertical spacing between reinforcement layers, and the required strength of the reinforcement.

Scope of Work

This specification details the technical and quality assurance requirements for furnishing all supervision, labor, material, equipment, and related services necessary to design and construct the load transfer platform.

Qualifications

The design/build Contractor constructing the load transfer platform shall have a minimum 3+ years experience installing geosynthetic reinforcement. References asserting this documentation should be provided to the Engineer a minimum of 30 days prior to construction.

Requirements

Site Preparation

The Contractor shall prepare subgrade and remove any deleterious materials. The foundation soil shall be observed and approved by the on-site Geotechnical Engineer prior to placement of select reinforced fill.

Reinforced Fill Materials

Reinforced fill material to be used in the load transfer platform shall consist of hard, durable aggregate. The gradation of the material shall be submitted to the State for review.

Geosynthetic Reinforcement

The Contractor shall submit a certificate stating that the reinforcement meets the design requirements for ultimate strength, creep, durability, installation damage, and coefficient of interaction for sliding in accordance with the design submittal.

Construction

The Contractor shall construct the column supported embankment in accordance with the approved plans.

Acceptance Criteria

A test section with a minimum of four rows of columns in each direction shall be constructed. The test embankment shall be installed in accordance with the Contractor's submitted plans. The geosynthetic reinforcement shall be installed and the embankment constructed. Settlement plates shall be installed to monitor settlement of the subgrade and settlement at the surface of the embankment. A surcharge load of ____ kN/m^2 (psf) shall be applied to the test embankment and settlements recorded. The test embankment will be considered acceptable if the measured surface settlement between columns, with the surcharge load in-place is less than ___ mm (in.).

Quality Assurance

Testing and Inspection

All compaction testing to determine specification compliance shall be provided by an independent testing agency or the Owner.

Measurement and Payment

Lump sum for all materials and labor to achieve specified criteria. *Performance specifications should delineate the area of the load transfer platform, as well as the criteria to be achieved.*

The test embankment should be paid on a lump sum basis.

CHAPTER 6
COST DATA

This chapter presents guidelines for preparing budget estimates in order that the economic feasibility of the load transfer platform portion of a column supported embankment may be assessed. The major benefit of the load transfer platform on the economics of a column supported embankment is the reduction of the required number of columns.

Estimating the cost of the load transfer platform is relatively straightforward. The components of the load transfer platform are the geosynthetic reinforcement, the select reinforced fill and the labor to install these materials. The geosynthetic reinforcement cost that has been used on several projects constructed between 1998 and 2002 varies between \$10.00-12.00/m^2 (\$8.35-10.00/yd^2). Select fill costs range from as low as \$0.75-2.00/kN (\$3.33-8.89/ton), depending on location and availability. The thickness of the load transfer platform may be estimated for preliminary cost purposes to be one-half the clear spacing between columns ((s-d)/2). The labor to construct the platform may be estimated to be 50% of the cost of the platform.

A preliminary cost estimate for a column spacing of 3 meters (10 ft.), with a column diameter of 0.5 meter (20-inch), and a unit cost of \$10/m^2 (\$8.35/yd^2) for the reinforcement and \$1.50/kN (\$6.66/ton) for the select fill is shown below:

Reinforcement cost per m^2 plan area of load transfer platform = \$10/m^2 (\$8.35/yd^2)

Select Fill Cost per m^2 plan area

Estimated thickness of platform (s-d)/2 = (3-0.5)/2 = 1.25 m

Estimated weight of select fill/m^2 plan area = (1.25 m)(20 kN/m^3) = 25 kN/m^2

Estimated cost of select fill/m^2 plan area (25 kN/m^2)(\$1.5/kN) = \$37.50/m^2 (\$31.35/yd^2)

Material costs = \$47.50/m^2 (\$39.75/yd^2)
Labor costs = \$47.50/m^2 (\$39.75/yd^2)

Total estimated cost for load transfer platform = \$ 95/m^2 (\$79.50/yd^2) of plan area of platform

Because of the multiple types of columns that are available, it is not practically feasible to provide cost guidelines for all of the column types.

CHAPTER 7
CASE HISTORIES

The following case history is a reprint of a paper for the 22nd World Road Congress.

Vibro-Concrete Columns and Geosynthetic Reinforced Load Transfer Platform Solve Difficult Foundation Problem

By

Lloyd W. Young Jr. & Mark N. Milton (Bechtel Infrastructure Corporation), James G. Collin (The Collin Group), and Eric Drooff (Hayward-Baker, Inc.)

Abstract: The embankment for a light rail line was to be constructed over soft compressible soils at a river crossing. Conventional approaches for constructing embankments over soft soils (i.e., wick drains and surcharge loading) were not viable on this project because of time constraints. The project owner's design-build contractor (Bechtel Infrastructure Corporation) recommended a new technology to improve the foundation soils. Vibro-Concrete columns (VCC), a sister technology to stone columns, was used to support the new embankment. The VCCs were designed based on a 7 to 9 foot triangular spacing. A geogrid reinforced Load Transfer Platform (LTP) was designed to transfer the embankment load to the VCCs. The embankment was retained along the length of the LTP with a concrete modular retaining wall system (i.e., T-Wall). This case history will review the reasons that soil improvement was selected for this project, design concepts, quality control measures used during construction of the VCCs, and post-construction performance.

1.0 INTRODUCTION

The new Rancocas Creek Railroad Bridge provides a high level crossing of Rancocas Creek to accommodate New Jersey Transit light-rail passenger trains as well as Conrail freight trains. The new bridge replaces an existing swing-span that had nearly stopped swinging due to age and had physically deteriorated to the point of near collapse. The new Rancocas Creek Bridge is one of the major structures for a new light rail line that extends from Camden to Trenton, New Jersey along the east shore of the Delaware River (Figure 1). The 34-mile-long Southern New Jersey Light Rail Transit System project will serve the communities in central New Jersey and provide improved transportation to Philadelphia and surrounding areas to the south and to Trenton and the rail systems extending to New York City to the north.

The project route follows the corridor of the old Camden & Amboy Railroad, which dates to the 1830's. While the alignment provided a predefined route, it also created numerous construction difficulties due to working within the constraints of an existing right-of-way and accommodating needed changes to the horizontal and vertical alignment to achieve the required passenger ride quality. This paper describes the design and construction of the foundations for the retaining wall system for the southern approach embankment to the Rancocas Bridge. Although this approach was in roughly the same horizontal location as the existing alignment, the profile was to be raised almost 20 feet, and replacing the existing single track with dual tracks also required widening of the existing embankment. The limited access to the work area and poor soil conditions required an innovative solution to providing the approach to the bridge.

Site Conditions

The project is located in the geologic units of the Atlantic Coastal Plain and is underlain by soils of the Potomac Formation. The Potomac, typically about 150 feet thick in this area, is characterized as a non-marine floodplain deposit and consists of moderately well graded gravelly sand. The surface of Potomac soils have been eroded by major streams that cross the area resulting in the later deposition of recent alluvial soils and the creation of bogs and marshes bordering the streams and estuaries.

The physical conditions along the approach to the Rancocas Bridge consisted of a one-hundred-year-old earth embankment with its top elevation slightly below Elevation + 10 feet. Although the embankment was initially built to hold two tracks, the erosion of the old embankment and a horizontal shift of the new track to increase the radius and improve riding quality for the light rail passengers caused the proposed alignment to "overhang" the old embankment to the east side and threaten the adjacent wetlands. (Figure 1)

The subsurface investigations in the project area were carried out in three separate phases starting during the initial design and continuing through the more detailed studies performed for the retaining wall alternative. A total of 29 borings were drilled with depths ranging up to 105 feet deep. The borings included standard penetration test sampling, undisturbed soil sampling and the installation of ground water observation wells. Ground water levels were found to be as high as El. +3.0 feet with corresponded tat the high tide level. Laboratory testing was performed on the recovered soil samples to help determine the engineering characteristics of the soils. The testing included: grain size, moisture content, Atterberg Limit, chemical analysis (pH, chloride, sulfate), organic content determination, triaxial strength and consolidation. The soil properties that were developed for the design are included in Table 4.

The subsurface investigation program identified a significant change in the quality of the near surface subsurface conditions as the alignment approached the swamp and marsh areas that bordered the south edge of Rancocas Creek. Approximately 800 feet south of the south edge of the creek, the soil profile consisted of 2 to 8 feet of manmade fill from the existing railroad embankment overlying a medium dense silty fine to medium sand with standard penetration resistances averaging about 18 blows per foot. These materials became dense to very dense below a depth ranging from 25 to 30 feet below the ground surface. From this point to the north, a sequence of recent alluvial layers started to be identified and these layers increased in total thickness to about 25 feet as the alignment approached the edge of the creek. These alluvial layers consisted of a layer of loose silty fine to medium sand overlying layers of peat, organic silt and silty clays. The laboratory test results showed that these materials were low in strength and highly compressible. The settlement and stability analyses showed that these materials could not carry the additional loading of a higher embankment without unacceptable settlement and the potential for slope stability failure.

Project Requirements

The initial design for the approach to the new Rancocas Bridge consisted of a seven span viaduct bridging over the soft compressible soils beneath the old embankment south of Rancocas Creek. The foundations for the viaduct consisted of concrete-filled pipe piles driven through the compressible soils into the underlying dense sands and gravel. The pile caps at each bent would carry precast concrete bridge beams for the support for the direct fixation track. This viaduct satisfied the project design requirements of providing stable support with essentially no settlement for the approach to the bridge and had the soil-to-structure transition for the track occurring out of the area of compressible soils. While important to highway construction, the need to prevent "bump at the end of the bridge" is critically important in railroad construction to prevent the settlement at this transition from causing excessive shear stresses from developing in the rails.

The evaluation of the construction sequence for the viaduct showed that each pile foundation and pile cap would need to be completely constructed in a sequential fashion starting at the south edge of the river. The pile caps would need to be built essentially above the existing grade, due to the presence of a fiber optic communications cable that could not be relocated, blocking access along the work site. The limited right-of-way width and the environmental restrictions did not allow construction of access roads adjacent to the work site to provide access. Additionally, the placement of the 80 to 100-ton bridge beams from the end of each span would require a 500-ton capacity crane, due to the large lifting radius for the 60-foot long beams. This combination of the construction sequence and equipment requirements caused the estimated schedule to be significantly longer and the costs higher than had been allocated in the budget. As a result, a value engineering study was started to provide an alternative to the viaduct.

Alternative Design and Construction Techniques

The value engineering study quickly identified that a retaining wall system would be required to keep the twin tracks of the approach within the limited right-of-way defined by the adjacent wetlands. A number of alternatives were evaluated based on a set of common requirements, including: a strong stable retaining structure and foundation essentially free from post-construction settlement, methods of construction that could be performed within a limited work area, and a design and construction budget equal or lower than the viaduct alternative. The evaluation of soil improvement methods ruled out wick drains and surcharge due to the rather long time required for stabilization and the amount of post construction settlement anticipated. Studies of retaining wall design and construction methods ruled out cast-in-place concrete construction due to cost and schedule, but did identify that a retaining wall system of large precast concrete elements was ideal for meeting stability requirements for supporting the light-rail and railroad freight loading.

Based on an evaluation of several retaining wall systems, the T-WALL Retaining Wall System was selected. The T-WALL system is a gravity system constructed of T-shaped precast concrete elements. The facing panels form the front plane of the wall and the ends of the stems of the "T" form the back plane (Figure 2). The system stability is a function of the weight of the concrete units and the select backfill between the stems. The stems need to be long enough at each level to develop a cross section to resist overturning and sliding at that level and to ensure soil/structure interaction. Internal stability is provided by the frictional forces created by the compaction of select backfill gripping the stem of the "T". Providing concrete shear keys to link the elements enhances the sliding resistance of the stacked precast units. The facing panels were 5'-0" wide and 2'-6" high and the stem lengths ranged from 14 feet long at the base of the wall to 10 feet at the top. The retaining wall forming the approach consisted of a 765-foot long east wall providing support to the tracks where the alignment was shifted to the east and off the top of the existing embankment. This east wall joined a 465-foot long ramp that ended at the abutment to the Rancocas Bridge. This portion of the retaining system was 35 feet wide at the top and had a maximum height of 20 feet. The weights of the precast elements were in the range of about 2,000 pounds making them easy to place with light lifting equipment.

Cost estimates were made and construction schedules were prepared to confirm that the ground improvement/retaining wall alternative offered advantages over the viaduct option. Approval was given to refine the selection of ground improvement methods and other aspects of the work.

2.0 SELECTION OF VIBRO-CONCRETE COLUMNS AND LOAD TRANSFER PLATFORM

Although the T-WALL retaining wall had tremendous flexibility to accommodate settlement due to the design and method of construction, the completed railroad track structure needed to be free from settlement to prevent excessive stress from developing in the rails. A settlement analysis of the proposed approach was performed using the characteristics of the unimproved ground to provide estimates of the total settlement. The maximum computed settlement was approximately 24 inches and it was expected that the duration of settlement would last for many years due to the continued compression of the peat soils.

Several ground improvement and deep foundation systems were considered including: driven piles, geopiers, stone columns, vibrated concrete columns, and cement deep soil mixing. Since each of these methods had unique and somewhat differing cost, schedule and performance characteristics, a performance specification was prepared that defined the objectives of the ground improvement. Information provided included: the physical layout of the completed walls, the design railroad loading of 2000 psf for the Cooper E80 loading with a 50 percent impact factor, and the settlement criteria for the completed wall. The settlement criteria was based on limiting the measured settlement at the base of the wall to approximately 1 inch over a 3-month period after the retaining wall was completed to full height. At the end of the three months, it was expected that the track structure and rails would be installed. Using this flexible criterion, it was expected that the contractor could take advantage of time to allow load distribution to take place within the improved ground while still providing a stable platform for the railroad.

Following evaluation of contractor proposals, Vibro-Concrete Columns (VCCs) were selected for the ground improvement method to support the retaining wall. The VCCs offered rapid construction schedule and also greater element stiffness, as compared to stone columns, when installed in the soft peat and clay soils. Although the traditional use of many ground improvement methods is to reduce settlement by replacing compressible soils with less compressible aggregate, the VCCs, constructed using 4000 pounds per square inch concrete, would be a combination of ground improvement and load transfer elements.

One of the key components required of this system was a means to transfer and distribute the load from the retaining wall to the VCCs. It was decided during design that the construction tolerances for the T-Wall and VCCs would not permit the direct support of the precast elements on the tops of the VCCs. Cost and schedule considerations did not permit the option of a reinforced concrete pile cap over the VCCs, and the stiffness of the pile cap would not permit load distribution to take place.

The solution was to design a geosynthetic reinforced load transfer platform to transfer the vertical load from the retaining wall system to the VCC. The design of the LTP used a Collin design method. This method is based on the creation of a stiff reinforced layer of soil that effectively transfers the loads from above the platform to the foundation system below the platform. The transfer of stress is achieved through a combination of soil arching and tension membrane theory. For this project, the clear span between VCC varied from 5-7 feet. The three foot thick LTP was required to transfer the load to the VCCs. The geosynthetic reinforcement was designed to carry the wedge of soil below the soil arch. Three layers of biaxial geogrid were required to create the platform. Figure 3 shows a typical cross-section.

Therefore, the elements were checked for structural capacity based on a loading of about 100 ton per column. This loading was computed using the railroad loading of 2,000 pounds per square foot, the dead weight of the T-WALL concrete sections and the dense graded aggregate backfill.

3.0 INSTALLATION OF THE VIBRO-CONCRETE COLUMNS

The VCC installation by Hayward-Baker Company went extremely well and no problems were encountered with penetration of the vibrator. The VCC spacing ranged from 7 to 9 feet and the columns were arranged in a triangular pattern. The 625 VCCs were installed in less than 3 weeks with maximum production reaching over 60 columns per day.

The use of pumped concrete allowed the materials to be delivered from behind the vibrator rig reducing congestion in the work area. Based on the allowable load capacity of 200 kips, a low slump 4,000 pounds per square inch pumpable concrete was selected and produced by ready mix suppliers certified by the State of New Jersey DOT. Because the concrete was pumped into the ground it was not air entrained, and did not use accelerators for winter conditions.

The columns were installed with a 19-inch diameter vibrator with an attached concrete pump line, permitting concrete to be pumped to the bottom of the vibrator. The expected diameter of the completed VCC was about 20 inches with a minimum size of about 16 inches due to some squeezing of the softer soils. The construction technique provided for longer vibrating times at the bottom and top of the columns to expand the base and top to as much as 30 inches. (A column expansion of about 24 inches was assumed for design.)

The VCC process makes use of a specially designed electrically driven bottom feed vibrator, which penetrates weak sub-soils to a level with either sufficient bearing capacity or to coarse grain soils which can be compacted and improved by the vibration. During the initial penetration of the vibrator, the weak cohesive and organic soils were displaced while granular layers were densified by the vibratory action. Once improvement of the load bearing formation was complete, the concrete pump was turned on, introducing concrete from the tip of the vibrator into the ground, and the column construction process was started by operating the vibrator to form a bulb of concrete at the base of the concrete column. Next the vibrator was slowly withdrawn while the concrete pressure was maintained, forming a continuous shaft of concrete up to ground level.

4.0 QUALITY CONTROL CONSIDERATIONS

Quality Control for VCCs, as with most deep foundation systems, consists of the selection of qualified personnel, materials testing and verification, adherence to established construction procedures, and load testing. While VCCs have not been used extensively in North America to date, however, with numerous contractors having employed vibro replacement systems such as stone columns under a great many projects over the past 25 years, skilled field personnel with hands on experience using depth vibrators are readily available. One of the most important aspects of quality control on the construction of VCCs is a strict adherence to the pre-assigned construction procedure described in the previous section. As such, properly trained and experienced personnel should be considered essential for the application of any such project. As an essential part of the process, each VCC was documented on a vibro concrete column record sheet which included, column location, start time, finish time, depth, maximum vibrator amperage, average pumping pressure, concrete volume pumped, and remarks of any changes or unusual observations

Production concrete strength was verified by casting four concrete cylinders daily in accordance with ASTM 31, cylinders where than tested in accordance with ASTM C39 at 5, 10 and 28 days. Once several production columns where completed in accordance with the established construction procedures, a load test column was selected to verify the adequacy of the VCC design.

The load test program consisted of statically load testing one column. The compression static load test was performed in general accordance with ASTM D1143-81 “Standard Test Method for Piles Under Static Axial Compression Load” and was begun after concrete break test indicated that the concrete had developed the required strength. Figure 4 shows the summary plots of load verses settlement. The load verses average test column butt deflection summarizes the average vertical movement of the test column butt at each load increment. The theoretical elastic compression line and the failure criteria (i.e. Davidson Criteria) are superimposed over the test results to illustrate the test column’s performance in relation to the theoretical compression line and the failure criteria at each load increment.

The compression load test was taken to a maximum test load of 300 kips or 150 percent of the design load. The gross displacement of the test column butt, 0.46 inches, was less than the specified failure criterion of 0.48 inches at 300 kips or 150 percent of the design compression load. The net set after unloading was 0.24 inches. Based on the results of the compression load test, the VCC satisfied the criteria for a design compression capacity of 200 kips.

5.0 POST-CONSTRUCTION PERFORMANCE

In order to provide confirmation of the criteria in the performance specification, settlement monitoring was performed on leveling pad installed at the base of the T-WALL. A series of readings along the east and west walls were taken after the completion of the T-WALL construction and the backfill in March of 2001. The next set of readings was taken three months later and showed a maximum settlement of about ¼-inch had occurred. This settlement was significantly less than the allowed settlement and provided the confirmation of the performance of the VCC’s and the load transfer platform.

The new Rancocas Bridge and the south approach have been in service for about one year. Additional settlement surveys taken after the one-year period show no additional deformation of the support systems.

6.0 CONCLUSIONS

The design and construction of the south approach to the Rancocas Bridge was accomplished using a variety of innovative elements that permitted some settlement during the construction process while providing a stable platform for the railroad at the end of construction. These elements were all selected to be rapidly installed in a work site 35 feet wide and over 1000 feet long with access only at one end using light construction equipment. Since the construction was over the winter months in New Jersey, the design elements minimized the use of concrete exposed to the cold and used select backfill that was not significant influence by the freezing weather.

These methods of construction are readily adaptable to highway as well as railroad construction where rapid construction over soft soils is necessary. The rapid construction can provide faster opening of the facility minimizing inconvenience to the public or businesses.

Table 4. Soil Properties Used for Design

Soil Layer	Depths Encountered	Average SPT N-Values	Shear Strength		Elastic Modulus	Allowable Bearing Capacity
			ϕ	c_u		
	(feet)	(bpf)	(deg)	(ksf)	(ksf)	(psf)
Existing Fill	2 – 8	9	28		300	2000
Silty, F-M Sands (Possible Fill)	2 – 20	18	30		700	2500
Peats, Organic Silts & Silty Clays	8 – 28	6		0.5	250	1500
Silty, F-M Sands	25 – 40	22	33		900	3500

Figure 1 – Site Location and Layout

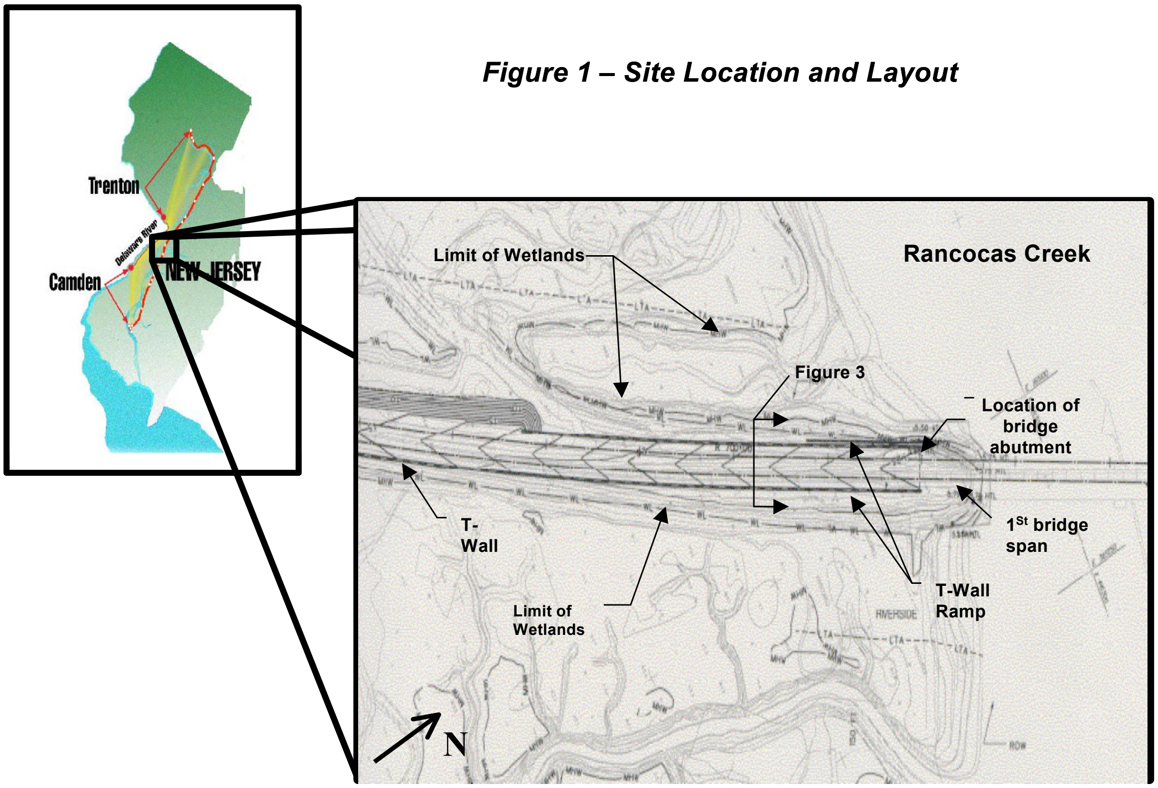

Figure 1 – Site Location and Layout

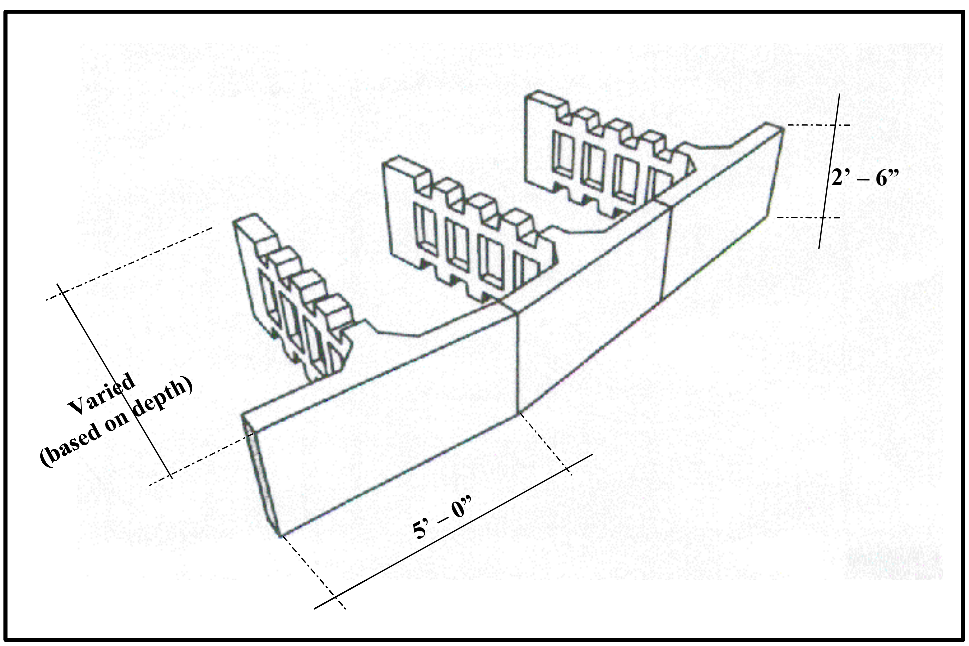

Figure 2 – T-Wall Block Diagram

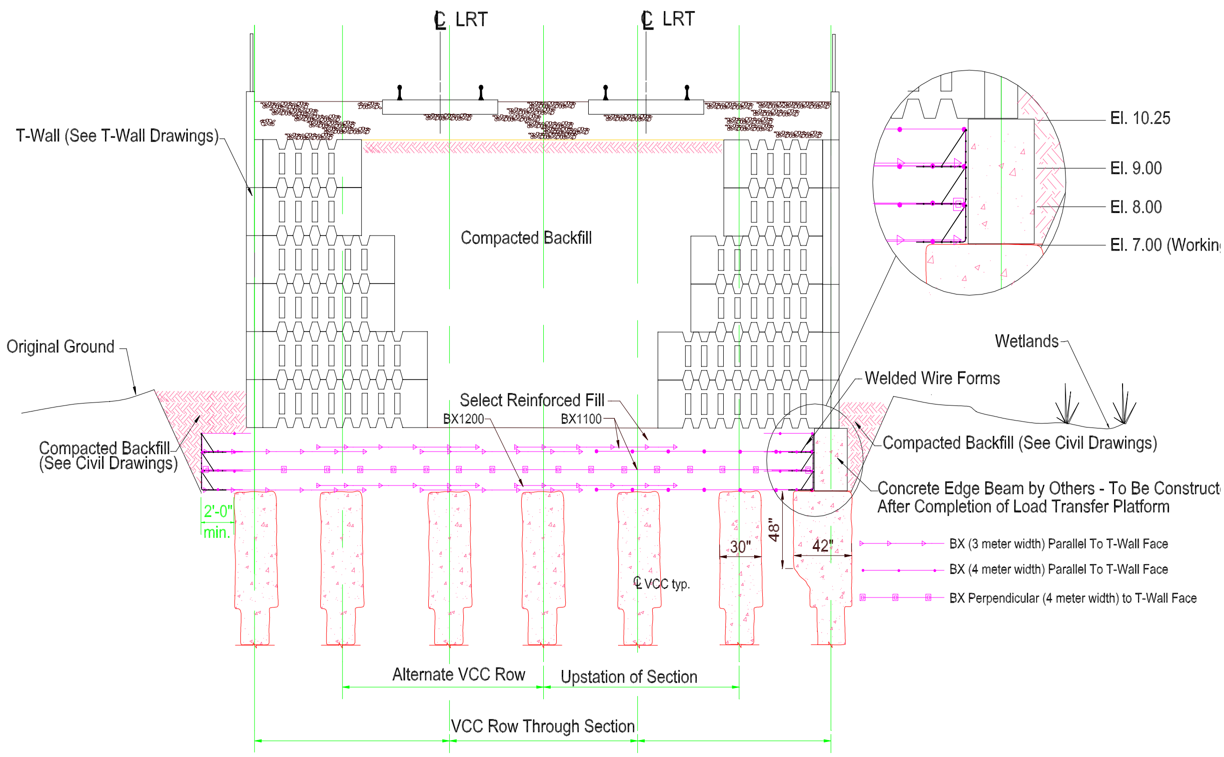

Figure 3. Typical Cross-Section.

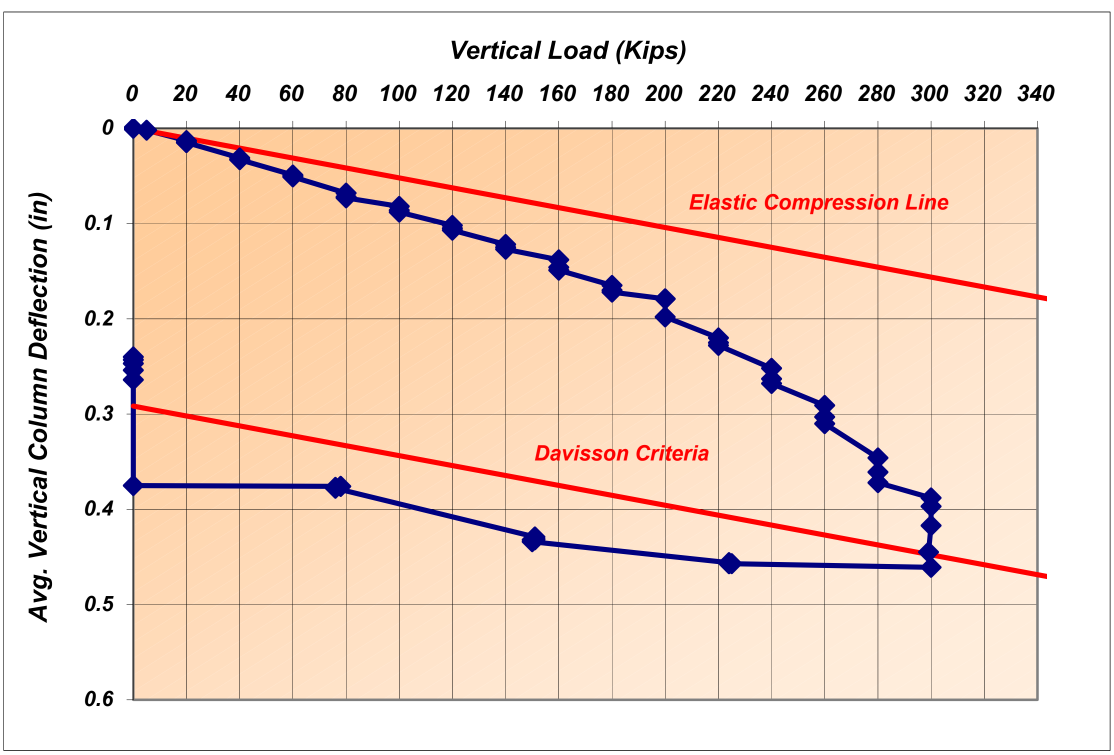

Figure 4. Vibro-Concrete Column Load Test Results.

Photograph 1. Vibro-Concrete Column Installation (Vibrator and Concrete Pump).

Photograph 2. Top of Finished Vibro-Concrete Column.

Photograph 3. T-Wall Installation.

Photograph 4. Completed T-Wall Ramp (Rancocas Bridge in background).

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Technical Summary # 8:

MECHANICALLY STABLIZED EARTH WALLS
And
REINFORCED SOIL SLOPES

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CHAPTER 1
DESCRIPTION AND HISTORY

1.1 PREFACE

Mechanically Stabilized Earth Walls (MSE) and Reinforced Soil Slopes (RSS) have been constructed worldwide for the last three decades. Applications, design methods, and construction specifications have evolved over this time span and have been detailed in the following documents that form the basis of this Technical Summary:

- *Mechanically Stabilized Earth Walls and Reinforced Soil Slopes, Design and Construction Guidelines*. FHWA-NHI-00-043, March 2001[(1)].

- *AASHTO Standard Specifications for Highway Bridges, 17th Edition* (2002) Section 5.8, Division I; and Section 7, Division II.

- *Corrosion/Degradation of Soil Reinforcements for Mechanically Stabilized Earth Walls and Reinforced Soil Slopes*, FHWA-NHI-00-044, September 2000 (Elias).

- *Geosynthetic Design and Construction Guidelines*. FHWA HI-95-038, May 1998.

1.2 DESCRIPTION

The improvement of soil by the addition of tensile reinforcements to form a stronger composite construction material is the basis for all reinforced soil construction methods. The most widespread application for transportation-related projects is in the construction of retaining structures and steepened soil slopes. The common components to both applications are reinforcements that may be either metallic or geosynthetic in a strip or grid configuration; a select backfill that is chiefly characterized by a restriction on its silt/clay content; a facing element that is usually a precast concrete panel for retaining wall applications; and a stabilized vegetated slope for steepened slope applications.

1.3 HISTORY

Inclusions have been used since prehistoric times to improve soil. The use of straw to improve the quality of adobe bricks dates back to earliest human history. Many primitive people used sticks and branches to reinforce mud dwellings. During the 17th and 18th centuries, French settlers along the Bay of Fundy in Canada used sticks to reinforce mud dikes. Some other early examples of man-made soil reinforcement include dikes of earth and tree branches, which have been used in China for at least 1,000 years and along the Mississippi River in the 1880s. Other examples include wooden pegs used for erosion and landslide control in England, and bamboo or wire mesh, used universally for revetment erosion control. Soil reinforcing can also be achieved by using plant roots.

The modern methods of soil reinforcement for retaining wall construction were pioneered by the French architect and engineer Henri Vidal in the early 1960s. His research led to the invention and development of Reinforced Earth®, a system in which steel strip reinforcement is used. The first wall to use this technology in the United States was built in 1972 on California State Highway 39, northeast of Los Angeles. In the last 25 years, more than 23,000 Reinforced Earth structures representing over 70 million m^2 (750 million $ft.^2$) of wall facing have been completed in 37 countries. More than 8,000 walls have been built in the United States since 1972. The highest wall constructed in the United States was on the order of 30 meters (98 feet).

Since the introduction of Reinforced Earth®, several other proprietary and nonproprietary systems have been developed and used. Table 1 provides a partial summary of some of the current systems by proprietary name, reinforcement type, and facing system.

Currently, most process patents covering soil-reinforced system construction or components have expired, leading to a proliferation of available systems or components that can be separately purchased and assembled by the erecting contractor. The remaining patents in force generally cover only the method of connection between the reinforcement and the facing.

For the first 20 years of use in the United States, an articulating precast facing unit 2 – 2.25 m^2 (21 – 24 $ft.^2$) generally square in shape, was the facing unit of choice. More recently, larger precast units of up to 5 m^2 (54 $ft.^2$) have been used, as have much smaller dry-cast units, generally in conjunction with geosynthetic reinforcements.

Table 1. Summary of reinforcement & face panel details for selected MSEW systems.

System Name	Reinforcement Detail	Typical Face Panel Detail[1]
Reinforced Earth The Reinforced Earth Company 2010 Corporate Ridge McLean, VA 22102	Galvanized Ribbed Steel Strips: 4 mm thick, 50 mm wide. Epoxy-coated strips also available.	Facing panels are cruciform shaped precast concrete 1.5 x 1.5 m x 140 mm thick. Half size panels used at top and bottom.
Retained Earth Foster Geotechnical 1600 Hotel Circle North San Diego, CA 92108-2803	Rectangular grid of W11 or W20 plain steel bars, 610 x 150 mm grid. Each mesh may have 4, 5, or 6 longitudinal bars. Epoxy-coated meshes also available.	Hexagonal and square precast concrete 1.5 x 1.5 m x 140 mm thick. Half size panels used at top and bottom.
Mechanically Stabilized Embankment Dept. of Transportation, Division of Engineering Services 5900 Folsom Blvd. P.O. Box 19128 Sacramento, CA 95819	Rectangular grid, nine 9.5 mm diameter plain steel bars on 610 x 150 mm grid. Two bar mats per panel (connected to the panel at four points).	Precast concrete; rectangular 3.81 m long, 610 mm high, 200 mm thick.
ARES Tensar Earth Technologies 5883 Glenridge Drive, Suite 200 Atlanta, GA 30328	HDPE Geogrid	Precast concrete panel; rectangular 2.74 m wide, 1.52 m high, 140 mm thick.
Welded Wire Wall Hilfiker Retaining Walls P.O. Drawer L Eureka, CA 95501	Welded steel wire mesh, grid 50 x 150 mm of W4.5 x W3.5, W9.5 x W4, W9.5 x W4, and W12 x W5 in 2.43 m wide mats.	Welded steel wire mesh, wrap around with additional backing mat 6.35 mm wire screen at the soil face (with geotextile or shotcrete, if desired).
Reinforced Soil Embankment Hilfiker Retaining Walls P.O. Drawer L Eureka, CA 95501	15 cm x 61 cm welded wire mesh: W9.5 to W20 - 8.8 to 12.8 mm diameter.	Precast concrete unit 3.8 m long, 610 mm high.
INTER-LOK Atlantic Concrete Industries P.O. Box 129 Tullytown, PA 19007	16 or 19 mm reinforcing steel bars fitted with 127 x 254 x 10 mm anchor plates and connected to a keyplate, and galvanized after fabrication.	Precast concrete panel; cross-shaped 1.83 m wide and 0.91 m high, 203 and 254 mm thick.
ISOGRID Neel Co. 6520 Deepford Street Springfield, VA 22150	Rectangular grid of W11 x W11 4 bars per grid.	Diamond shaped precast concrete units, 1.5 x 2.5 m, 140 mm thick.
MESA Tensar Earth Technologies, Inc. 5883 Glenridge Drive, Suite 200 Atlanta, GA 30328	HDPE Geogrid	MESA HP (high performance), DOT[3] OR Standard units (203 mm high by 457 mm long face, 275 mm nominal depth). (dry cast concrete)
PYRAMID The Reinforced Earth® Company 2010 Corporate Ridge McLean, VA 22102	Galvanized WWM, size varies with design requirements or Grid of PVC coated, Polyester yarn (Matrex Geogrid).	Pyramid® unit (200 mm high x 400 mm long face, 250 mm nominal depth) (dry cast concrete).
Maccaferri Terramesh System Maccaferri Gabions, Inc. 43A Governor Lane Blvd. Williamsport, MD 21795	Continuous sheets of galvanized double twisted woven wire mesh with PVC coating.	Rock filled gabion baskets laced to reinforcement.
Strengthened Earth Gifford-Hill & Co. 2515 McKinney Ave. Dallas, Texas 75201	Rectangular grid, W7, W9.5 and W14, transverse bars at 230 and 450 mm.	Precast concrete units, rectangular or wing shaped, 1.82 m x 2.13 m x 140 mm.
MSE Plus SSL 4740 Scotts Valley Drive Scotts Valley, CA 95066	Rectangular grid with W11 to W24 longitudinal bars and W11 transverse. Mesh may have 4 – 6 longitudinal bars spaced at 200 mm.	Rectangular precast concrete panels 1.5 m high, 1.82 m wide, with a thickness of 152 or 178 mm.
KeySystem - Inextensible Keystone Retaining Wall Systems 4444 W. 78th Street Minneapolis, MN 55435	Galvanized welded wire ladder mat of W7.5 to W17 bars with crossbars at 150 – 600 mm.	KeySystem concrete facing unit is 203 mm high x 457 mm wide x 305 mm deep (dry cast concrete).

Table 1 (cont'd).
Summary of reinforcement & face panel details for selected MSEW systems

System Name	Reinforcement Detail	Typical Face Panel Detail[1]
KeySystem - Extensible Keystone Retaining Wall Systems 4444 W. 78th Street Minneapolis, MN 55435	Stratagrid high-tenacity knit polyester geogrid soil reinforcement by Strata Systems, Inc. PVC coated.	Keystone Standard and Compac concrete facing units are 203 mm high x 457 mm wide x 457 mm or 305 mm deep (dry cast concrete).
Tricon System Tricon Precast Ltd. 15055 Henry Road Houston, TX 77060	Galvanized welded-wire.	Rectangular precast concrete panels with a face area of 4.2 sq. m.
Versa-Lok Retaining Wall Systems 6348 Highway 36 Blvd. Oakdale, MN 55128	PVC coated PET or HDPE geogrids.	Versa-Lok concrete unit 152 mm high x 406 mm long x 305 mm deep (dry cast concrete)
Anchor Wall Systems 5959 Baker Road Minnetonka, MN 55345	PVC coated PET geogrid.	Anchor Landmark concrete unit 381 mm high x 203 mm long x 302 (small unit) or 321(large unit) mm deep (dry cast concrete).
[1]Additional facing types are possible with most systems.		

The use of geotextiles in MSE walls and RSS started after the beneficial effect of reinforcement with geotextiles was noticed in highway embankments over weak subgrades. The first geotextile-reinforced wall was constructed in France in 1971, and the first structure of this type in the United States was constructed in 1974. Since about 1980, the use of geotextiles in reinforced soil has increased significantly.

Geogrids for soil reinforcement were developed around 1980. The first use of geogrid in earth reinforcement was in 1981. Extensive use of geogrid products in the United States started in about 1983, and they now comprise a growing portion of the market.

The first reported use of reinforced steepened slopes is believed to be the west embankment for the great wall of China. The introduction and economy of geosynthetic reinforcements has made the use of steepened slopes economically attractive. A survey of usage in the mid-1980s identified several hundred completed projects. At least an order of magnitude more RSS structures have been constructed since that study. The highest constructed RSS structure in the U.S. to date has been 43 m (140 ft.).

A representative list of geosynthetic manufacturers and suppliers is shown in Table 2.

Table 2. Representative list of geotextile and geogrid manufacturers and suppliers.

Amoco Fabrics and Fibers Co. 260 The Bluff Austelle, GA 30168	BBA Nonwovens - Reemay, Inc. 70 Old Hickory Blvd. Old Hickory, TN 37138	Carthage Mills 4243 Hunt Road Cincinnati, OH 45242
Colbond Geosynthetics (Akzo) 1301 Sand Hill Road Enka, NC 28728	Contech Construction Products 1001 Grove Street Middletown, OH 45044	Huesker, Inc. 11107 A S. Commerce Blvd. Charlotte, NC 28273
Luckenhaus North America 175 Mehler Lane Martinsville, VA 24112	Maccaferri Inc. 10303 Governor Lane Blvd. Williamsport, MD 21795	Ten Cate Nicolon 365 S. Holland Drive Pendegrass, GA 30567
SI Geosolutions 4019 Industry Drive Chattanooga, TN 37416	Strata Systems, Inc. 380 Dahlonega Hwy, Ste. 200 Cummings, GA 30040	SynTeen Technical Fabrics 1950 W. Meeting Street Lancaster, SC 29721
Tenax Corporation 4800 East Monument Street Baltimore, MD 21205	Tensar Earth Technologies 5883 Glenridge Drive, Suite 200 Atlanta, GA 30328	WEBTEC Inc. 2619 West Blvd. Charlotte, NC 28219
[1] List is from the Geosynthetic Materials Association membership list.		

Current Usage

It is believed that MSE walls have been constructed in every state in the United States. Major users include transportation agencies in Georgia, Florida, Texas, Pennsylvania, New York, and California, which rank among the largest road-building states.

It is estimated that more than 700,000 m^2 (7,500,000 ft.^{2}) of MSE retaining walls with precast facing are constructed, on average, every year in the United States, which may represent more than half of all retaining wall usage for transportation applications.

The majority of the MSE walls for permanent applications either constructed to date or presently planned use segmental precast concrete facing and galvanized steel reinforcements. The use of geotextile faced MSE walls in permanent construction has been limited to date. They are quite useful for temporary construction, where more extensive use has been made.

Recently, modular block dry cast facing units have gained acceptance due to their lower cost and nationwide availability. These small concrete units are generally mated with grid reinforcement, and the wall system is referred to as modular block wall (MBW). It has been reported that more than 200,000 m^2 (2,000,000 $ft.^2$) of MBW walls have been constructed yearly in the United States, when considering all types of transportation related applications. The current yearly usage for transportation-related applications is estimated at about 50 projects per year.

The use of RSS structures has expanded dramatically in the last decade, and it is estimated that several hundred RSS structures have been constructed in the United States. Currently, 70 – 100 RSS projects are being constructed yearly in connection with transportation related projects in the United States, with an estimated projected vertical face area of 130,000 m^2/year (1,400,000 $ft.^2$/yr.).

1.4 FOCUS AND SCOPE

The purpose of this technical summary is to acquaint the reader with soil reinforcement technology by providing an overview and indicating areas of application. The items discussed include

- areas of application
- advantages/disadvantages
- feasibility evaluations
- design concepts and methods
- bidding methods, construction control, and material specifications
- case histories
- cost data

This document does not provide in-depth recommendations on design, construction control, and long-term monitoring. These items are detailed in FHWA-NHI-00-043, *Mechanically Stabilized Earth Walls and Reinforced Soil Slopes, Design and Construction Guidelines*.

CHAPTER 2
DESIGN CONSIDERATIONS

MSEW structures are cost-effective alternatives for most applications where reinforced concrete or gravity type walls have traditionally been used to retain soil. These include bridge abutments and wing walls, as well as areas where the right-of-way is restricted, such that an embankment or excavation with stable side slopes cannot be constructed. They are particularly suited to economical construction in steep-sided terrain, in ground subject to slope instability, or in areas where foundation soils are poor.

MSE walls offer significant technical and cost advantages over conventional reinforced concrete retaining structures at sites with poor foundation conditions. In such cases, the elimination of costs for foundation improvements, such as piles and pile caps, that may be required for support of conventional structures, have resulted in cost savings of greater than 50% on completed projects.

Some additional successful uses of MSE walls include

- Temporary structures, which have been especially cost-effective for temporary detours necessary for highway reconstruction projects.

- Reinforced soil dikes, which have been used for containment structures for water and waste impoundments around oil and liquid natural gas storage tanks. (The use of reinforced soil containment dikes is economical and can also result in savings of land because a vertical face can be used, which reduces construction time.)

- Dams and seawalls, including increasing the height of existing dams.

- Bulk materials storage using sloped walls.

Representative uses of MSE walls for various applications are shown in Figures 1 and 2.

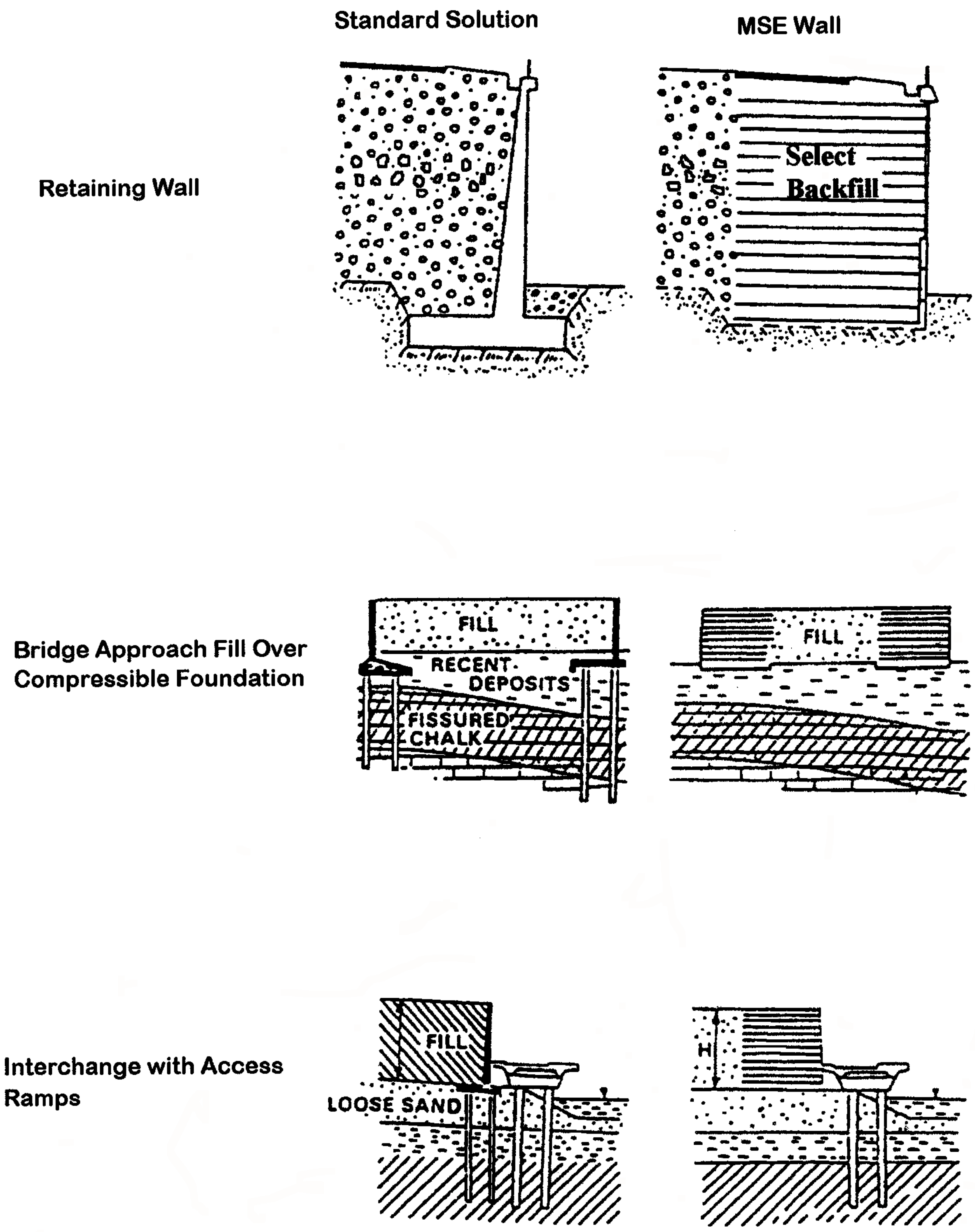


Figure 1. MSE Wall Urban Applications.

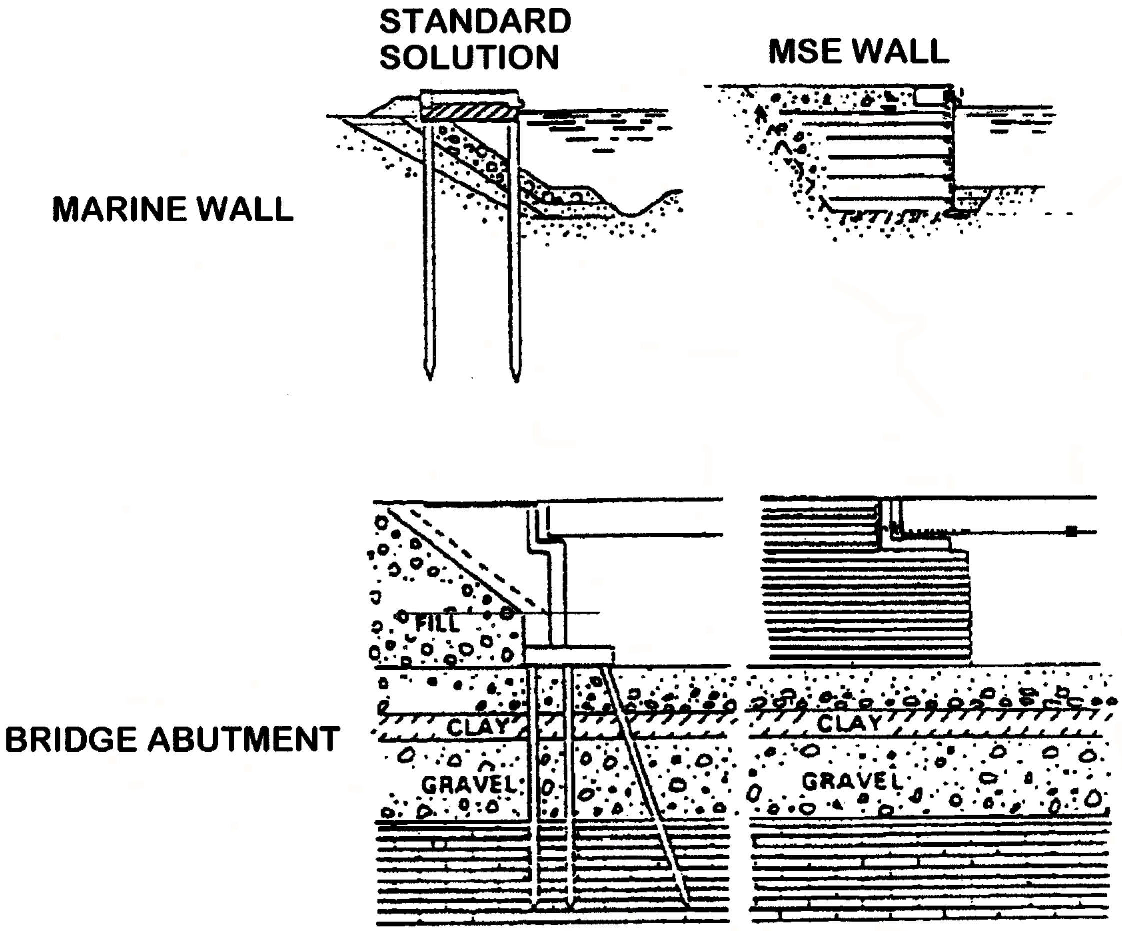


Figure 2. MSE Wall Applications, Abutments.

Reinforced Soil Slopes are cost-effective alternatives for new construction where the cost of fill, right-of-way, and other considerations may make a steeper slope desirable. However, even if foundation conditions are satisfactory, slopes may be unstable at the desired slope angle. Existing slopes, natural or manmade, may also be unstable, as is painfully obvious when they fail. As shown in Figure 3, multiple layers of reinforcement may be placed in the slope during construction or reconstruction to reinforce the soil and provide increased slope stability. Reinforced slopes are a form of mechanically stabilized earth that incorporate planar reinforcing elements in constructed earth sloped structures with face inclinations of less than 70 degrees. Typically, geosynthetics are used for reinforcement.

There are two primary purposes for using reinforcement in engineered slopes.

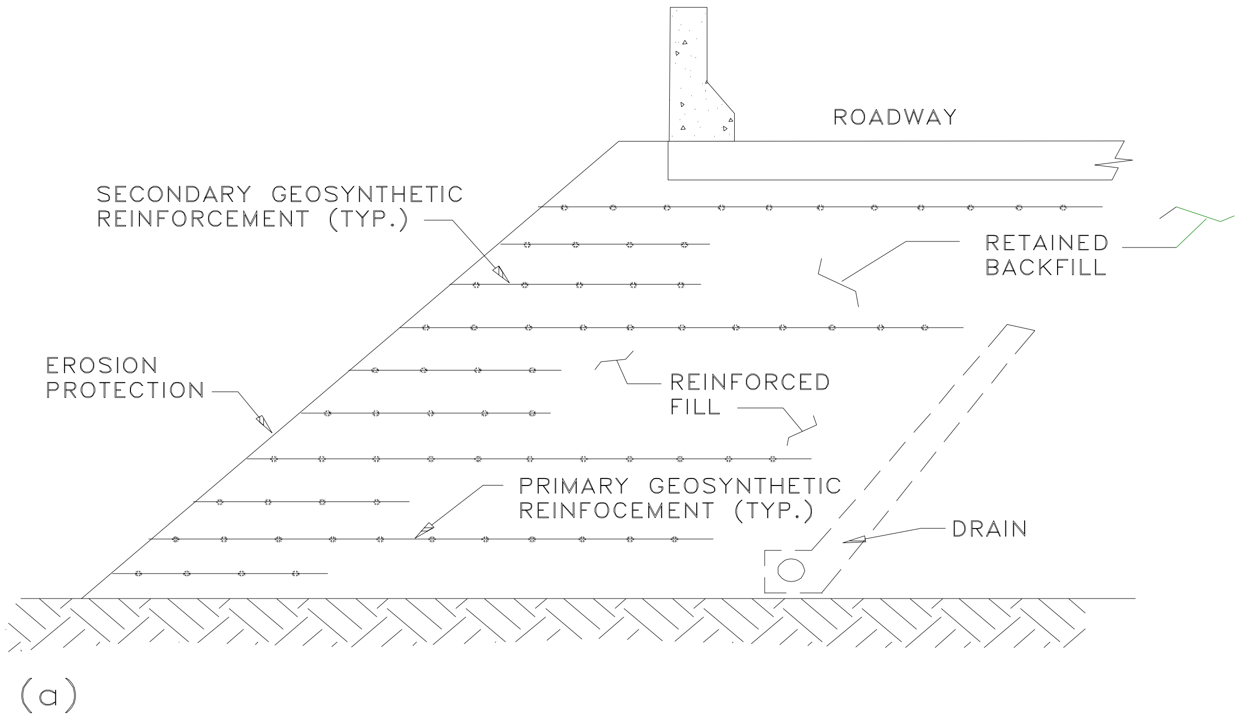
- To increase the stability of the slope, particularly if a steeper than *safe* unreinforced slope is desirable or after a failure has occurred, as shown in Figure 3a.

- To provide improved compaction at the edges of a slope, thus decreasing the tendency for surface sloughing as shown in Figure 3b.

The principal purpose for using reinforcement is to construct an RSS embankment at an angle steeper than could otherwise be safely constructed with the same soil. The increase in stability allows for construction of steepened slopes on firm foundations for new highways and as replacements for flatter unreinforced slopes and retaining walls. Roadways can also be widened over existing flatter slopes without encroaching on existing right-of-ways. In the case of repairing a slope failure, the new slope will be safer, and reusing the slide debris rather than importing higher quality backfill may result in substantial cost savings. These applications are illustrated in Figure 4.

The second purpose for using reinforcement is at the edge of a compacted fill slope to provide lateral resistance during compaction. The increased lateral resistance allows for an increase in compacted soil density over that normally achieved and provides increased lateral confinement for the soil at the face. Even modest amounts of reinforcement in compacted slopes have been found to prevent sloughing and reduce slope erosion. Edge reinforcement also allows compaction equipment to more safely operate near the edge of the slope.

Further compaction improvements have been found in cohesive soils through the use of geosynthetics with in-plane drainage capabilities (e.g., nonwoven geotextiles) that allow for rapid pore pressure dissipation in the compacted soil.



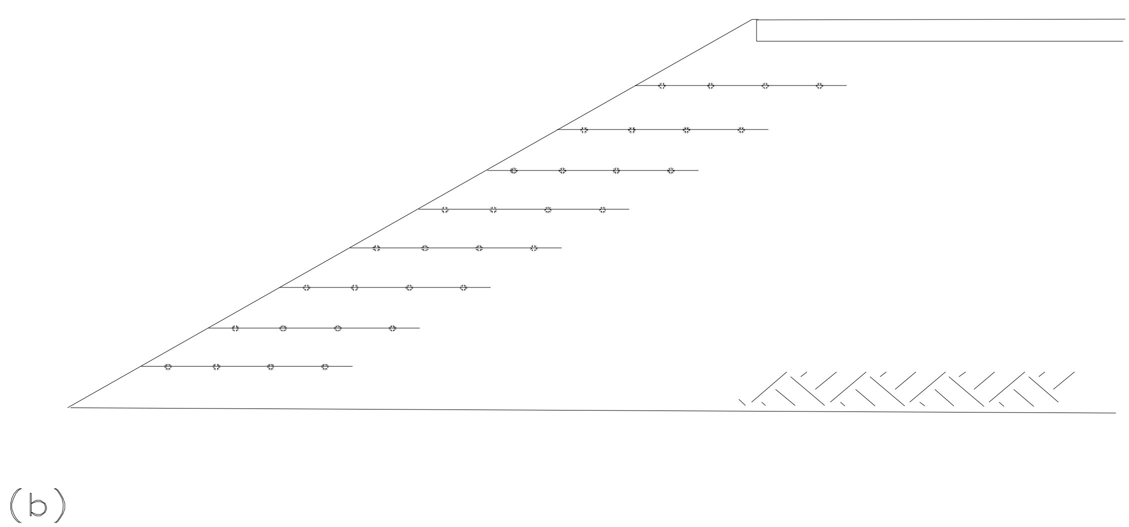


Figure 3. Slope Reinforcement using Geosynthetics to Provide Slope Stability.

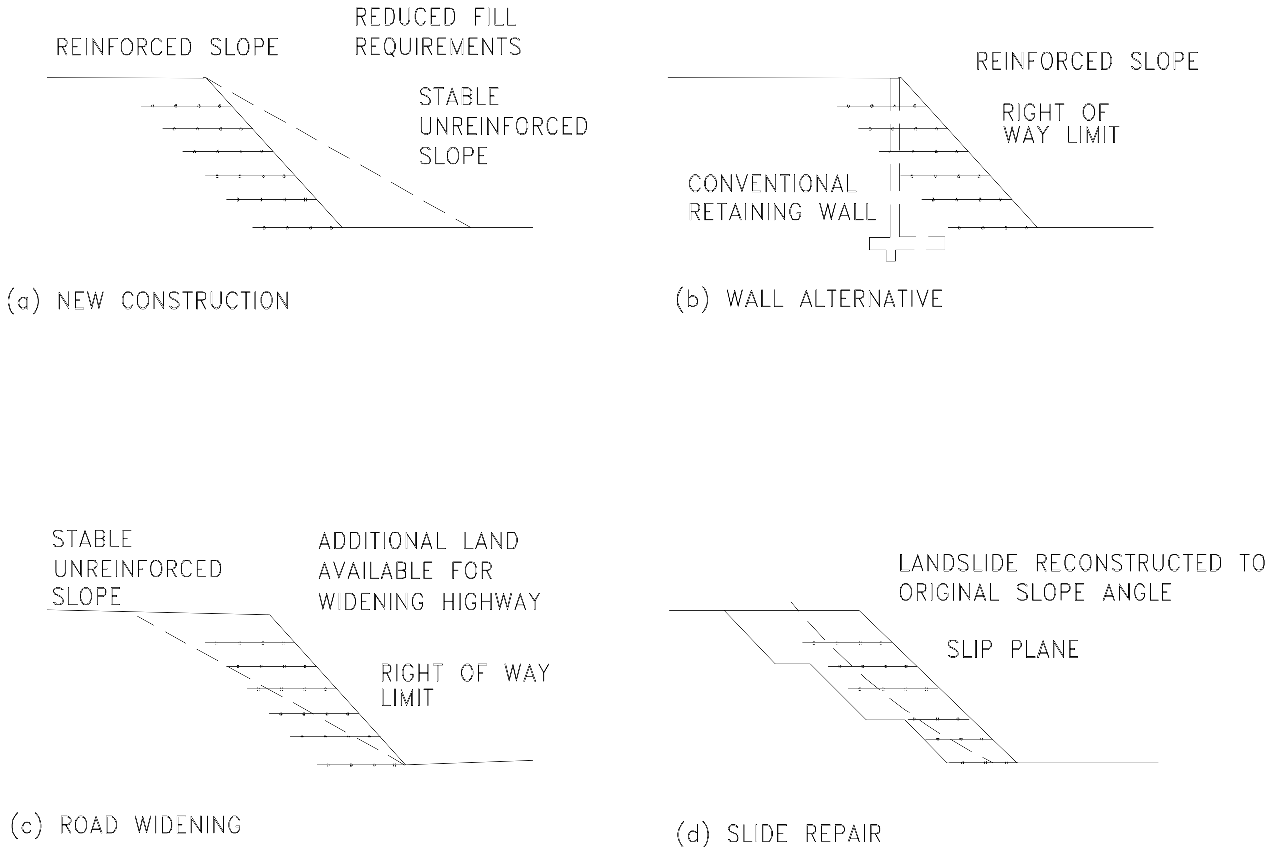


Figure 4. Application of Reinforced Soil Slopes.

Compaction aids placed as intermediate layers between reinforcement in steepened slopes may also be used to provide improved face stability and to reduce layers of more expensive, primary reinforcement as shown in Figure 3a.

Other applications of reinforced slopes have included

- upstream/downstream face improvements to increased height of dams
- permanent levees
- temporary flood control structures
- decreased bridge spans
- temporary road widening for detours
- prevention of surface sloughing during periods of saturation
- embankment construction with wet, fine-grained soils

2.2 ADVANTAGES AND DISADVANTAGES

a. Advantages of Mechanically Stabilized Earth (MSE) Walls

MSE walls have many advantages compared with conventional reinforced concrete and concrete gravity retaining walls. MSE walls

- Use simple and rapid construction procedures and do not require large construction equipment.
- Do not require experienced craftsmen with special skills for construction.
- Require less site preparation than other alternatives.
- Need less space in front of the structure for construction operations.
- Reduce right-of-way acquisition.
- Do not need rigid, unyielding foundation support because MSE structures are tolerant to deformations.
- Are cost effective.
- Are technically feasible to heights in excess of 25 m (80 ft.).

The relatively small quantities of manufactured materials required, rapid construction, and competition among the developers of different proprietary systems have resulted in a cost reduction relative to traditional types of retaining walls. MSE walls are likely to be more economical than other wall systems for walls higher than about 3 m (10 ft.) or where special foundations would be required for a conventional wall.

One of the greatest advantages of MSE walls is their flexibility and capability to absorb deformations due to poor subsoil conditions in the foundations. Also, based on observations in seismically active zones, these structures have demonstrated a higher resistance to seismic loading than have rigid concrete structures.

Precast concrete facing elements for MSE walls can be made with various shapes and textures (with little extra cost) for aesthetic considerations. Masonry units, timber, and gabions also can be used with advantage to blend in the environment.

b. Advantages of Reinforced Soil Slopes (RSS)

The economic advantages of constructing a safe, steeper RSS than would normally be possible are the resulting material and right-of-way savings. It also may be possible to decrease the quality of materials required for construction. For example, in repair of landslides it is possible to reuse the slide debris rather than to import higher quality backfill. Right-of-way savings can be a substantial benefit, especially for road widening projects in urban areas where acquiring new right-of-way is always expensive and, in some cases, unobtainable. RSS also provide an economical alternative to retaining walls. In some cases, reinforced slopes can be constructed at about one-half the cost of MSEW structures.

The use of vegetated-faced reinforced soil slopes that can be landscaped to blend with natural environments may also provide an aesthetic advantage over retaining wall type structures. However, there are some potential maintenance issues that must be addressed, such as mowing grass-faced, steep slopes. However, these can be satisfactorily handled in design.

In terms of performance, due to inherent conservatism in the design of RSS, they are actually safer than flatter slopes designed at the same factor of safety. As a result, there is a lower risk of long-term stability problems developing in the slopes. Such problems often occur in compacted fill slopes that have been constructed to low factors of safety and/or with marginal materials (e.g., deleterious soils, such as shale, fine grained low cohesive silts, plastic soils, etc.). The reinforcement may also facilitate strength gains in the soil over time from soil aging and though improved drainage, further improving long-term performance.

c. Disadvantages

The following general disadvantages may be associated with all reinforced soil structures:

- They require a relatively large space behind the wall or outward face to obtain enough wall width for internal and external stability.

- MSEW require select granular fill. (At sites where there is a lack of granular soils, the cost of importing suitable fill material may render the system uneconomical.) Requirements for RSS are typically less restrictive.

- Suitable design criteria are required to address corrosion of steel reinforcing elements, deterioration of certain types of exposed facing elements such as geosynthetics by ultra violet rays, and potential degradation of polymer reinforcement in the ground.

- Since design and construction practice of all reinforced systems are still evolving, specifications and contracting practices have not been fully standardized, especially for RSS.

- The design of soil-reinforced systems often requires a shared design responsibility between material suppliers and owners, and greater input from agencies' geotechnical specialists in a domain often dominated by structural engineers.

2.3 FEASIBILITY EVALUATION

The major factors that influence the selection of an MSEW/RSS alternative for any project include

- geologic and topographic conditions
- environmental conditions
- size and nature of the structure
- aesthetics
- durability considerations
- performance criteria
- availability of materials
- experience with a particular system or application
- cost

Many MSEW systems have proprietary features. Some companies provide services, including design assistance, preparation of plans and specifications for the structure, supply of the manufactured wall components, and construction assistance.

The various wall systems have different performance histories, and this sometimes creates difficulty in obtaining adequate technical evaluation. Some systems are more suitable for permanent walls, others are more suitable for low walls, and some are applicable for remote areas, while others are more suited for urban areas. The selection of the most appropriate system will thus depend on the specific project requirements.

RSS embankments have been constructed with a variety of geosynthetic reinforcements and treatments of the outward face. These factors again may create an initial difficulty in adequate technical evaluation. A number of geosynthetic reinforcement suppliers provide design services, as well as technical assistance during construction.

Issues focused on selection factors are summarized in this chapter. Technical issues are summarized in Chapters 3 and 4.

a. Geologic and Topographic Conditions

MSE structures are particularly well-suited where a "fill type" wall must be constructed or where side-hill fills are indicated. Under the latter condition, the volume of excavation may be small, and the general economy of this type of construction is not jeopardized.

The adequacy of the foundation to support the fill weight must be determined as a first-order feasibility evaluation.

Where soft compressible soils are encountered, preliminary stability analyses must be made to determine if sufficient shear strength is available to support the weight of the reinforced fill. As a rough first approximation for vertically faced MSE structures, the available shear strength must be equal to at least 2.0 to 2.5 times the weight of the fill structure. For RSS embankments, the required foundation strength is somewhat less and dependent on the actual slope considered.

Where these conditions are not satisfied, ground improvement techniques must be considered to increase the bearing capacity at the foundation level. These techniques include but are not limited to

- excavation and removal of soft soils and replacement with a compacted structural fill
- use of lightweight fill materials
- in-situ densification by dynamic compaction, or improvement by use of surcharging with or without wick drains
- construction of stone columns

Where marginal to adequate foundation strength is available, preliminary settlement analyses should be made to determine the potential for differential settlement, both longitudinally along a proposed structure, as well as transversely to the face. This second-order feasibility evaluation is useful in determining the appropriate type of facing systems for MSE walls and in planning appropriate construction staging to accommodate the settlement.

In general, concrete faced MSEWs using discrete articulating panels can accommodate maximum longitudinal differential settlements of about 1/100, without the introduction of special sliding joints between panels. Full-height concrete panels are considerably less tolerant and should not be considered where differential settlements are anticipated.

The performance of reinforced soil slopes generally is not affected by differential longitudinal settlements.

b. Environmental Conditions

The primary environmental condition affecting reinforcement type selection and potential performance of MSE structures is the aggressiveness of the in-situ ground regime that can cause deterioration to the reinforcement. Post-construction changes must be considered where de-icing salts or fertilizers are subsequently used.

For steel reinforcements, in-situ regimes containing chloride and sulfate salts generally in excess of 200 PPM accelerate the corrosive process, as do acidic regimes characterized by a pH of less than 5.[2] Alkaline regimes characterized by pH > 10 will cause accelerated loss of galvanization. Under these conditions, bare steel reinforcements could be considered.

Certain in-situ regimes have been identified as being potentially aggressive for geosynthetic reinforcements. Polyesters (PET) degrade in highly alkaline or acidic regimes. Polyolefins appear to degrade only under certain highly acidic regimes.[2]

A secondary environmental issue is site accessibility, which may dictate the nature and size of the facing for MSEW construction. Sites with poor accessibility or remote locations may lend themselves to lightweight facings, such as metal skins; modular blocks (MBW) that could be erected without heavy lifting equipment; or the use of geotextile or geogrid wrapped facings and vegetative covers.

RSS construction with an organic vegetative cover must be carefully chosen to be consistent with native perennial cover that would establish itself quickly and would thrive with available site rainfall.

c. Size and Nature of Structure

Theoretically, there is no upper limit to the height of MSEW that can be constructed. Structures in excess of 25 m (80 ft.) have been successfully constructed with steel reinforcements, although such heights for transportation-related structures are rare. RSS embankments have been constructed to greater heights.

Practical limits are often dictated by economy, available ROW, and the tensile strength of commercially available soil reinforcing materials. For bridge abutments, there is no theoretical limit to the span length that can be supported, although the longer the span, the greater is the area of footing necessary to support the beams. Since the allowable bearing capacity in the reinforced fill is usually limited to 200 kPa (4000 psf), a large abutment footing further increases the span length, adding cost to the superstructure. This additional cost must be balanced by the potential savings of the MSE alternate to a conventional abutment wall, which would have a shorter span length. As an option in such cases, it might be economical to consider support of the bridge beams on deep foundations, placed within the reinforced fill zone.

The lower limit to height is usually dictated by economy. When used with traffic barriers, low walls on good foundations of less than 3 – 4 m (10 – 13 ft.) are often uneconomical, as the cost of the overturning moment leg of the traffic barrier approaches one-third of the total cost of the MSE structure in place. For cantilever walls, the barrier is simply an extension of the stem with a smaller impact on overall cost.

The total size of structure (square meters of face) has little impact on economy compared with other retaining wall types. However, the unit cost for small projects of less than 300 m^2 (3,000 $ft.^2$) is likely to be 10 – 15% higher.

RSS may be cost effective in rural environments where ROW restrictions exist or on widening projects where long sliver fills are necessary. In urban environments, they should be considered where ROW is available, as they are always more economical than vertically faced MSEW structures.

d. Aesthetics

Precast concrete facing panels may be cast with an unlimited variety of texture and color for an additional premium that seldom exceeds 15% of the facing cost, which, on average, would mean a 4 – 6% increase on total in-place cost.

Modular block wall facings are often comparable in cost to precast concrete panels, except on small projects (less than 400 m^2 (4,000 $ft.^2$)) where the small size introduces savings in erection equipment costs or where irregular geometry requires that special, made-to-order concrete panels are cast to fit. MBW facings may be manufactured in color and with a wide variety of surface finishes.

The outward face treatment of RSS, generally is by vegetation, which is initially more economical than the concrete facing used for MSE structures. However, maintenance costs may be considerably higher, and the long-term performance of many outward face treatments has not been established.

e. Performance Criteria

The performance criteria considered during selection should be limited to estimating the effects of deflections both horizontal and vertical (settlement) to determine potential impact on the selection of facing elements.

Typically, lateral MSE wall displacement occurs during construction and is primarily a function of the extensibility of the reinforcement. Therefore, the impact of greater horizontal displacement when using geosynthetic reinforcement, may be mitigated during construction by increasing the back batter of facing panels. MSE structures using precast concrete panels have significant deformation tolerance both longitudinally along a wall and perpendicularly to the front face. Therefore, poor foundation conditions seldom preclude their use. However, where significant differential settlement is anticipated (more than 1/100), sufficient joint width and/or slip joints must be provided to preclude panel cracking. This factor may influence the type and design of the facing panel selected.

Square panels generally adapt to larger longitudinal differential settlements better than long rectangular panels of the same surface area. Guidance on minimum joint width and limiting differential settlements that can be tolerated is presented in Table 3.

MSE walls constructed with full height panels should be limited to differential settlements of 1/500. Walls with drycast facing (MBW) should be limited to settlements of 1/200. For walls with welded wire facings, the limiting differential settlement should be 1/50.

Table 3. Relationship between joint width and limiting differential settlements for MSE precast panels.

Joint Width	Limiting Differential Settlement
20 mm	1/100
13 mm	1/200
6 mm	1/300

Where significant differential settlement perpendicular to the wall face is anticipated, the reinforcement connection may be overstressed. Where the back of the reinforced soil zone will settle more than the face, the reinforcement could be placed on a sloping fill surface, which is higher at the back end of the reinforcement to compensate for the greater vertical settlement. This may be the case where a steep surcharge slope is constructed. This latter construction technique, however, requires that surface drainage be carefully controlled after each day's construction. Alternatively, where significant differential settlements are anticipated, ground improvement techniques may be warranted to limit the settlements, as outlined in geological conditions.

2.4 LIMITATIONS

The current AASHTO *Interim Specifications for Highway Bridges*, indicates that MSE walls should not be used under the following conditions:

- When utilities other than highway drainage must be constructed within the reinforced zone where future access for repair would require the reinforcement layers to be cut. A similar limitation should be considered for RSS structures.

- With galvanized metallic reinforcements exposed to surface or ground water contaminated by acid mine drainage or other industrial pollutants, as indicted by low pH and high chlorides and sulfates.

- When floodplain erosion may undermine the reinforced fill zone, or where the depth to scour cannot be reliably determined.

In addition, selection of the facing type chosen for RSS structures should be consistent with the recommendations contained in Table 6. Further, for vegetated facings, the site climatic conditions must be able to support growth.

CHAPTER 3
CONSTRUCTION SYSTEMS, MATERIALS, AND METHODS

Since the expiration of the fundamental process and concrete facing panel patents obtained by the Reinforced Earth Co. for MSEW systems and structures, the engineering community has adopted the generic term "*Mechanically Stabilized Earth*" to describe this type of retaining wall construction.

Trademarks, such as Reinforced Earth®, Retained Earth®, MESA® etc., describe systems with some present or past proprietary features or unique components marketed by nationwide commercial suppliers. Other trademark names appear yearly to differentiate systems marketed by competing commercial entities that may include proprietary or novel components, or for special applications.

A system for either MSEW or RSS structures is defined as a complete supplied package that includes design, specifications, and all **prefabricated** materials of construction necessary for the complete construction of a soil reinforced structure. Often, technical assistance during the planning and construction phase is also included. Components marketed by commercial entities for integration by the owner in a coherent system are not classified as systems.

3.1 SYSTEMS DESCRIPTION

MSEW/RSS systems can be described by the reinforcement geometry, soil-to-reinforcement stress transfer mechanism, reinforcement material, extensibility of the reinforcement material, and the type of facing and connections.

Structure Type

Two functional types of structures can be considered:

- **Mechanically Stabilized Earth Wall** (MSEW) structures are characterized by a vertical or near vertical facing (i.e., ≥ 70°), usually with a precast or drycast concrete panel for permanent applications.

- **Reinforced Soil Slopes** (RSS) embankment structures are characterized by an inclined face between 35 – 70°, with a vegetative facing for slopes flatter than 1:1 and armored for steeper inclinations.

Reinforcement Geometry

Three types of reinforcement geometry can be considered:

- **Linear unidirectional**. Strips, including smooth or ribbed steel strips, or coated geosynthetic strips over a load-carrying fiber.

- **Composite unidirectional**. Grids or bar mats characterized by grid spacing greater than 150 mm (6 in.).

- **Planar bidirectional**. Continuous sheets of geosynthetics, welded wire mesh, and woven wire mesh. The mesh is characterized by element spacing of less than 150 mm (6 in).

Reinforcement Material

Distinction can be made between the characteristics of metallic and nonmetallic reinforcements:

- **Metallic reinforcements**. Typically of mild steel. The steel is usually galvanized or may be epoxy coated.

- **Nonmetallic reinforcements**. Generally, polymeric materials consisting of polypropylene, polyethylene, or polyester.

Reinforcement Extensibility

There are two classes of extensibility:

- **Inextensible**. The deformation of the reinforcement at failure is much less than the deformability of the soil.

- **Extensible**. The deformation of the reinforcement at failure is comparable to or even greater than the deformability of the soil.

The performance and durability considerations for these two classes of reinforcement vary considerably and are detailed in the MSEW and RSS Design and Construction course reference manuals[1, 2].

3.2 FACING SYSTEMS

The types of facing elements used in the different MSE systems control their aesthetics because they are the only visible parts of the completed structure. A wide range of finishes and colors can be provided in the facing. In addition, the facing provides protection against backfill sloughing and erosion, and provides a drainage path in certain cases. The type of facing influences settlement tolerances. Major facing types are:

- **Segmental precast concrete panels,** summarized in Table 1 and illustrated in Figure 5. The precast concrete panels have a minimum thickness of 140 mm (5-½ in.) and are of a cruciform, square, rectangular, diamond, or hexagonal geometry. Temperature and tensile reinforcement are required, but will vary with the size of the panel. Vertically adjacent units are usually connected with shear pins.

- **Dry cast modular block wall (MBW) units**. These are relatively small, squat concrete units that have been specially designed and manufactured for retaining wall applications. The mass of these units commonly ranges from 15 – 50 kg (30 – 110 lbs), with units of 35 – 50 kg (75 – 110 lbs) routinely used for highway projects. Unit heights typically range from 100 – 200 mm (4 – 8 in.) for the various manufacturers. Exposed face length usually varies from 200 – 450 mm (8 – 18 in.). Nominal width (dimension perpendicular to the wall face) of units typically ranges between 200 – 600 mm (8 – 24 in.). Units may be manufactured solid or with cores. Full height cores are filled with aggregate during erection. Units are normally dry-stacked (i.e., without mortar) and in a running bond configuration. Vertically adjacent units may be connected with shear pins, lips, or keys. They are referred to by trademarked names, such as Keystone®, Versa-Lok®, Allan®, etc. They are illustrated in Figure 6.

- **Metallic Facings.** The original Reinforced Earth® system had facing elements of galvanized steel sheet formed into half cylinders. Although precast concrete panels are now commonly used in Reinforced Earth walls, metallic facings may be appropriate in structures where difficult access or difficult handling requires lighter facing elements.

- **Welded Wire Grids.** Wire grid can be bent up at the front of the wall to form the wall face. This type of facing is used in the Hilfiker, Tensar, and Reinforced Earth wire retaining wall systems.

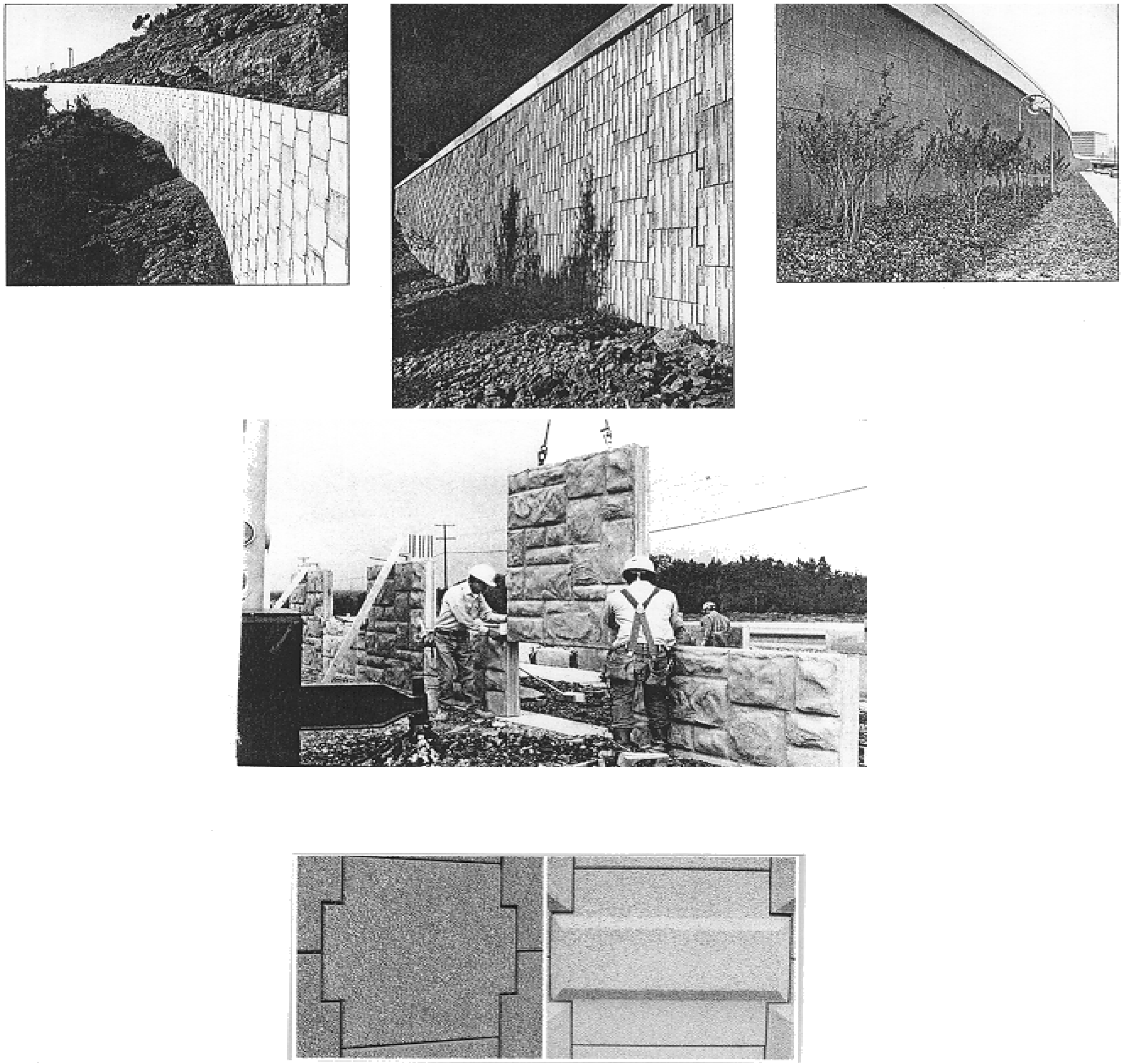


Figure 5. MSE Wall Surface Textures.

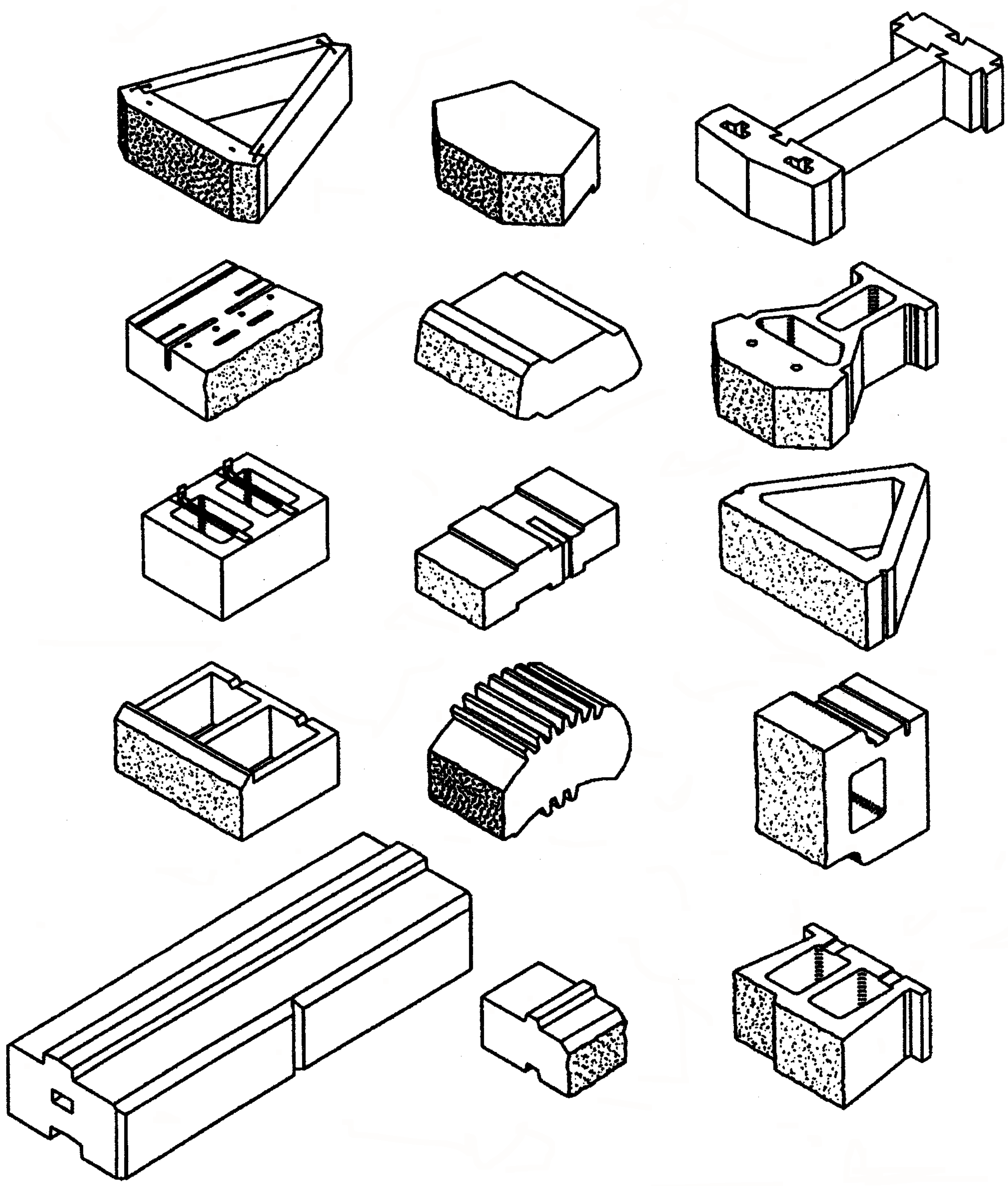


Figure 6. Examples of Commercially Available MBW Units
(from Design Manual for Segmental Retaining Walls[11]).

- **Gabion Facing.** Gabions (rock-filled wire baskets) can be used as facing with reinforcing elements consisting of welded wire mesh, welded bar-mats, geogrids, geotextiles, or the double-twisted woven mesh placed between or connected to the gabion baskets.

- **Geosynthetic Facing.** Various types of geotextile reinforcement are looped around at the facing to form the exposed face of the retaining wall. These faces are susceptible to ultraviolet light degradation, vandalism, and damage due to fire. Alternately, a geosynthetic grid used for soil reinforcement can be looped around to form the face of the completed retaining structure in a similar manner to welded wire mesh and fabric facing. Vegetation can grow through the grid structure and can provide both ultraviolet light protection for the geogrid and a pleasing appearance.

- **Post-Construction Facing.** For wrapped faced walls, the facing – whether geotextile, geogrid, or wire mesh – can be attached after construction of the wall by shotcreting, guniting, cast-in-place, or attaching prefabricated facing panels made of concrete, wood, or other materials. This multi-staging facing approach adds cost, but is advantageous where significant settlement is anticipated.

Precast elements can be cast in several shapes and provided with facing textures to match environmental requirements that blend aesthetically into the environment. Retaining structures using precast concrete elements as the facings can have surface finishes similar to any reinforced concrete structure.

Retaining structures with metal facings have the disadvantage of shorter life because of corrosion unless provision is made to compensate for it.

Facings using welded wire or gabions have the disadvantages of an uneven surface, exposed backfill materials, more tendency for erosion of the retained soil, possible shorter life from corrosion of the wires, and more susceptibility to vandalism. These disadvantages can, of course, be countered by providing shotcrete or by hanging facing panels on the exposed face and compensating for possible corrosion. The greatest advantages of such facings are low cost, ease of installation, design flexibility, good drainage (depending on the type of backfill) that provides increased stability, and possible treatment of the face for vegetative and other architectural effects. The facing can easily be adapted and well blended with natural country environment. These facings, as well as geosynthetic wrapped facings, are especially advantageous for construction of temporary or other structures with a short-term design life.

Dry cast modular block MBW facings may raise some concerns as to durability in aggressive freeze-thaw environments when produced with water absorption capacity significantly higher than that of wet-cast concrete. Historical data provide little insight, as their usage history is less than two decades. Further, because the cement is not completely hydrated during the dry cast process, (as is often evidenced by efflorescence on the surface of units), a highly alkaline regime may establish itself at or near the face area, and may limit the use of some geosynthetic products as reinforcements. Freeze-thaw durability is enhanced for products produced at higher compressive strengths and low water absorption ratios.

The outward faces of slopes in RSS structures are usually vegetated if 1:1 or flatter. The vegetation requirements vary by geographic and climatic conditions and are, therefore, project specific. Slopes steeper than approximately 1:1 typically require facing support during construction. Exact slope angles will vary with soil types, i.e., amount of cohesion. Removable facing supports (e.g., wooden forms) or left-in-place welded wire mesh forms are typically used. Facing support may also serve as permanent or temporary erosion protection, depending on the requirements of the slope.

3.3 REINFORCEMENT TYPES

Most, although not all, systems using precast concrete panels use steel reinforcements that are typically galvanized, but may be epoxy coated. Two types of steel reinforcements are in current use:

- **Steel strips.** The currently commercially available strips are ribbed top and bottom, 50 mm (2 in.) wide and 4 mm (5/32-in.) thick. Smooth strips 60 – 120 mm (2⅜ – 4¾-in.) wide, 3 – 4 mm (⅛ – 5/32-in.) thick have been used.

- **Steel grids.** Welded wire grid using 2 to 6 W7.5 to W24 longitudinal wire spaced at either 150 – 200 mm (6 – 8 in.). The transverse wire may vary from W11 to W20 and are spaced based on design requirements from 230 – 600 mm (9 – 24 in.). Welded steel wire mesh spaced at 50 by 50 mm (2 by 2-in.) of thinner wire has been used in conjunction with a welded wire facing. Some MBW systems use steel grids with 2 longitudinal wires.

Most MBW systems use geosynthetic reinforcement, principally geogrids. The following types are widely used and available:

- **High Density Polyethylene (HDPE) geogrid.** These are of uniaxial manufacture and are available in up to 6 styles, differing in strength.

- **PVC coated polyester (PET) geogrid.** Available from a number of manufacturers. They are characterized by bundled high-tenacity PET fibers in the longitudinal load carrying direction. For longevity, the PET is supplied as a high molecular weight fiber and is further characterized by a low carboxyl end group number.

- **Geotextiles.** High strength geotextiles can be used principally in connection with reinforced soil slope (RSS) construction. Both polyester (PET) and polypropylene (PP) geotextiles have been used.

3.4 REINFORCED BACKFILL MATERIALS

MSE Structures

MSE walls require high quality backfill for durability, good drainage, constructability, and good soil reinforcement interaction, which can be obtained from well graded, granular materials. Many MSE systems depend on friction between the reinforcing elements and the soil. In such cases, a material with high friction characteristics is specified and required. Some systems rely on passive pressure on reinforcing elements, and, in those cases, the quality of backfill is still critical. These performance requirements generally eliminate soils with high clay contents.

From a reinforcement capacity point of view, lower quality backfills could be used for MSEW structures; however, a high quality granular backfill has the advantages of being free draining, providing better durability for metallic reinforcement, and requiring less reinforcement. There are also significant handling, placement, and compaction advantages in using granular soils. These include an increased rate of wall erection and improved maintenance of wall alignment tolerances.

The following requirements are consistent with current practice:

- **Select Granular Fill Material for the Reinforced Zone**. All backfill material used in the structure volume for MSEW structures shall be reasonably free from organic or other deleterious materials and shall conform to the following gradation limits as determined by AASHTO T-27.

1. **Gradation Limits**

Sieve Size	Percent Passing
102 mm (4 in.)[a]	100
0.425 mm (No. 40)	0-60
0.075 mm (No. 200)	0-15

Plasticity Index (PI) shall not exceed 6.

[a]In order to apply default F* (see Chapter 4) values, C_u should be greater than or equal to 4. As a result of recent research on construction survivability of geosynthetics and epoxy coated reinforcements, it is recommended that the maximum particle size for these materials be reduced to 19 mm (¾-inch) for geosynthetics and epoxy and PVC coated reinforcements unless tests are or have been performed to evaluate the extent of construction damage anticipated for the specific fill material and reinforcement combination.

2. **Soundness.** The materials shall be substantially free of shale or other soft, poor durability particles. The material shall have a magnesium sulfate soundness loss (or a sodium sulfate value less than 15% after five cycles) of less than 30% after four cycles. Testing shall be in accordance with AASHTO T-104.

The fill material must be free of organic matter and other deleterious substances, as these materials not only enhance corrosion but also result in excessive settlements. The compaction specifications should include a specified lift thickness and allowable range of moisture content with reference to optimum. The compaction requirements of backfill are different in close proximity to the wall facing (within 1.5 – 2 m). Lighter compaction equipment is used near the wall face to prevent buildup of high lateral pressures from the compaction and to prevent facing panel movement. Because of the use of this lighter equipment, a backfill material of good quality in terms of both friction and drainage, such as crushed stone, is recommended close to the face of the wall to provide adequate strength and tolerable settlement in this zone. It should be noted that granular fill containing even a few percent fines may not be free draining, and drainage requirements should always be carefully evaluated.

The design of buried steel elements of MSE structures is predicated on backfills exhibiting minimum or maximum electrochemical index properties, and then designing the structure for maximum corrosion rates associated with these properties. These recommended index properties and their corresponding limits are shown in Table 4.

Reinforced fill soils must meet the indicated criteria to be qualified for use in MSEW construction using steel reinforcements.

Where geosynthetic reinforcements are planned, the limits for electrochemical criteria would vary depending on the polymer. Limits based on current research are shown in Table 5. They would be equally applicable to RSS structures.

Table 4. Recommended limits of electrochemical properties for backfills when using steel reinforcement.

Property	Criteria	Test Method
Resistivity	>3000 ohm-cm	AASHTO T-288-91
pH	>5<10	AASHTO T-289-91
Chlorides	<100 PPM	AASHTO T-291-91
Sulfates	<200 PPM	AASHTO T-290-91
Organic Content	1% max.	AASHTO T-267-86

Table 5. Recommended limits of electrochemical properties for backfills when using geosynthetic reinforcements.

Base Polymer	Property	Criteria	Test Method
Polyester (PET)	pH	>3<9	AASHTO T-289-91
Polyolefin (PP & HDPE)	pH	>3	AASHTO T-289-91

RSS Structures

For RSS structures, less select backfill can be used, as facings are typically flexible and can tolerate some distortion during construction. Even so, a high quality embankment fill meeting the following gradation requirements to facilitate compaction and minimize reinforcement requirements is recommended. The following guidelines are provided as recommended backfill requirements for RSS construction:

Sieve Size	Percent Passing
20 mm* (¾-in.)	100
4.76 mm (No. 4)	100 - 20
0.425 mm (No. 40)	0 - 60
0.075 mm (No. 200)	0 - 50

Plasticity Index (PI) ≤ 20 (AASHTO T-90)

Soundness: Magnesium sulfate soundness loss less than 30% after 4 cycles, based on AASHTO T-104, or equivalent sodium sulfate soundness of less than 15% after 5 cycles.

** The maximum fill size can be increased (up to 100 mm) provided field tests have been or will be performed to evaluate potential strength reduction due to construction damage. In any case, geosynthetic strength reduction factors for site damage should be checked in relation to the maximum particle size to be used and the angularity of the larger particles.*

MSE and RSS Structures

Backfill compaction should be based on 95% of AASHTO T-99, and ±2% of optimum moisture, w_{opt}.

The reinforced fill criteria outlined above represent materials that have been successfully used throughout the United States and have resulted in excellent structure performance. Peak shear strength parameters are used in the analysis. For MSE walls, a lower bound frictional strength of 34 degrees would be consistent with the specified fill, although some nearly uniform fine sands meeting the specifications limits may exhibit friction angles of 31 – 32 degrees. Higher values may be used if substantiated by laboratory direct shear or triaxial test results for the site-specific material used or proposed. However, extreme caution is advised for use of friction angles above 40 degrees for design, due to a lack of field performance data and questions concerning mobilization of shear strength above that value.

Fill materials outside of these gradation and plasticity index requirements have been used successfully; however, problems including significant distortion and structural failure have also been observed. While there may be a significant savings in using lower quality backfill, property values must be carefully evaluated with respect to influence on both internal and external stability. For MSE walls constructed with reinforced fill containing more than 15% passing a 0.075 mm (#200) sieve and/or the PI exceeds 6, both total and effective shear strength parameters should be evaluated in order to obtain an accurate assessment of horizontal stresses, sliding, compound failure (behind and through the reinforced zone), and the influence of drainage on the analysis. Both long-term and short-term pullout tests, as well as soil/reinforcement interface friction tests, should be performed. Settlement characteristics must be carefully evaluated, especially in relation to downdrag stresses imposed on connections at the face and settlement of supported structures. Drainage requirements at the back, face, and beneath the reinforced zone must be carefully evaluated (e.g., use flow nets to evaluate influence of seepage forces and hydrostatic pressure).

For RSS structures, where a considerably greater percentage of fines (minus #200 sieve) is permitted, lower bound values of frictional strength equal to 28 to 30 degrees would be reasonable for the backfill requirements listed. A significant economy could again be achieved if laboratory direct shear or triaxial test results on the proposed fill are performed, justifying a higher value. Likewise, soils outside the gradation range listed should be carefully evaluated and monitored.

3.5 MISCELLANEOUS MATERIALS OF CONSTRUCTION

Walls using precast concrete panels require bearing pads in their horizontal joints to provide some compressibility and movement between panels and to preclude concrete-to-concrete contact. These materials are either neoprene, SBR rubber, or HDPE.

All joints are covered with a polypropylene (PP) geotextile strip to prevent the migration of fines from the backfill. The compressibility of the horizontal joint material should be a function of the wall height. Walls with heights greater than 15 m (50 ft.) may require thicker or more compressible joints to accommodate the larger vertical loads due to the weight of panels in the lower third of the structure.

3.6 CONSTRUCTION SEQUENCE

The following is an outline of the principal sequence of construction for MSEW and RSS. Specific systems, special appurtenances, and specific project requirements may vary from the general sequence indicated.

a. Construction of MSEW Systems with Precast Facings

The construction of MSEW systems with a precast facing is carried out as follows:

- **Preparation of subgrade**. This step involves removal of unsuitable materials from the area to be occupied by the retaining structure. All organic matter, vegetation, slide debris, and other unstable materials should be stripped off and the subgrade compacted.

  In unstable foundation areas, ground improvement methods, such as dynamic compaction, stone columns, wick drains, or other foundation stabilization/ improvement methods, would be completed prior to wall erection.

- **Placement of a leveling pad for the erection of the facing elements**. This generally unreinforced concrete pad is often only 300 mm wide (1 ft.) and 150 mm (6 in.) thick and is used for MSEW construction only, where concrete panels are subsequently erected. A gravel pad has been often substituted for MBW construction.

  The purpose of this pad is to serve as a guide for facing panel erection and is not intended as a structural foundation support.

- **Erection of the first row of facing panels on the prepared leveling pad**. Facings may consist of either precast concrete panels, metal facing panels, or dry cast modular blocks.

  The first row of facing panels may be full, or half-height panels, depending upon the type of facing used. The first tier of panels must be shored up to maintain stability and alignment. For construction with modular dry-cast blocks, full sized blocks are used throughout with no shoring.

  The erection of facing panels and placement of the soil backfill proceed simultaneously.

- **Placement and compaction of backfill on the subgrade to the level of the first layer of reinforcement and its compaction**. The fill should be compacted to the specified density, usually 95 – 100% of AASHTO T-99 maximum density and within the specified range of optimum moisture content. Compaction moisture contents dry of optimum are recommended.

  A key to good performance is *consistent* placement and compaction. Wall fill lift thickness must be controlled, based on specification requirements and vertical distribution of reinforcement elements. The uniform loose lift thickness of the reinforced backfill should not exceed 300 mm (12 in.). Reinforced backfill should be dumped into, or parallel to, the rear and middle of the reinforcement and bladed toward the front face. Random fill placement behind the reinforced volume should proceed simultaneously.

- **Placement of the first layer of reinforcing elements on the backfill**. The reinforcements are placed and connected to the facing panels when the compacted fill has been brought up to the level of the connection. They are generally placed perpendicular to back of the facing panels. Refer to Chapter 9 of FHWA NHI-00-043 *Mechanically Stabilized Earth Walls and Reinforced Soil Slopes Design and Construction Guidelines* for more detailed construction controls procedures.

- **Placement of the backfill over the reinforcing elements to the level of the next reinforcement layer and compaction of the backfill**. The previously outlined steps are repeated for each successive layer.

- **Construction of traffic barriers and copings**. This final construction sequence is undertaken after the final panels have been placed, and the backfill has been completed to its final grade.

  A complete sequence is illustrated in Figures 7 through 9.

b. Construction of RSS Systems

As the reinforcement layers are easily incorporated between the compacted lifts of fill, construction of reinforced slopes is very similar to normal slope construction. The elements of construction consist of simply

- placing the soil

- placing the reinforcement
- constructing the face

The usual construction sequence is shown in Figure 10.

- **Site Preparation**. Consists of successively clearing and grubbing, leveling and proof-rolling the subgrade, prior to the placement of the first level of reinforcement.

- **Reinforcing Layer Placement**. The reinforcement should be placed with the principal strength direction perpendicular to the face of the slope and secured with retaining pins to prevent movement during fill placement. A minimum overlap of 150 mm (6 in.) is recommended along the edges perpendicular to the slope for wrapped face structures. Alternatively, with geogrid reinforcement, the edges may be clipped or tied together. When geosynthetics are not required for face support, no overlap is required, and edges should be butted.

- **Reinforcement Backfill Placement**. Place fill to the required lift thickness on the reinforcement using a front-end loader or dozer, operating on previously placed fill or natural ground. Maintain a minimum of 150 mm (6 in.) of fill between the reinforcement and the wheels or tracks of construction equipment.

- **Compaction**. Compact with a vibratory roller or plate type compactor for granular materials or a rubber-tired or smooth drum roller for cohesive materials. When placing and compacting the backfill material, care should be taken to avoid deformation or movement of the reinforcement. Use lightweight compaction equipment near the slope face to help maintain face alignment. Provide close control on the water content and density of the backfill. It should be compacted to at least 95% of the standard AASHTO T99 maximum density, within 2% of optimum moisture. If the backfill is a coarse aggregate, then a relative density test should be used.

- **Face Construction**. If slope facing is required to prevent sloughing (i.e., slope angle β is greater than ϕ_{soil}) or erosion, several options are available. Sufficient reinforcement lengths could be provided for wrapped faced structures. A face wrap may not be required for slopes up to 1H:1V, if the reinforcement is maintained at close spacing (i.e., every lift or every other lift, but no greater than 400 mm (16 in.)). In this case, the reinforcement can be simply extended to the face. For this option, a facing treatment (see Facing Options in Chapter 4) should be applied at sufficient intervals during construction to prevent face erosion.

The following procedures are recommended for wrapping the face.

- Turn up reinforcement at the face of the slope and return the reinforcement a minimum of 1 m (3 ft.) into the embankment below the next reinforcement layer (see Figure 10).

- For steep slopes, formwork may be required to support the face during construction, especially if lift thicknesses of 450 – 600 mm (18 – 24 in.) or greater are used.

- For geogrids, a fine mesh screen or geotextile may be required at the face to retain backfill materials.

More detailed information on the construction sequence, inspection, and potential monitoring programs for both MSE and RSS structures can be found in FHWA-NHI-00-043 *Mechanically Stabilized Earth Walls and Reinforced Soil Slopes Design and Construction Guidelines.*

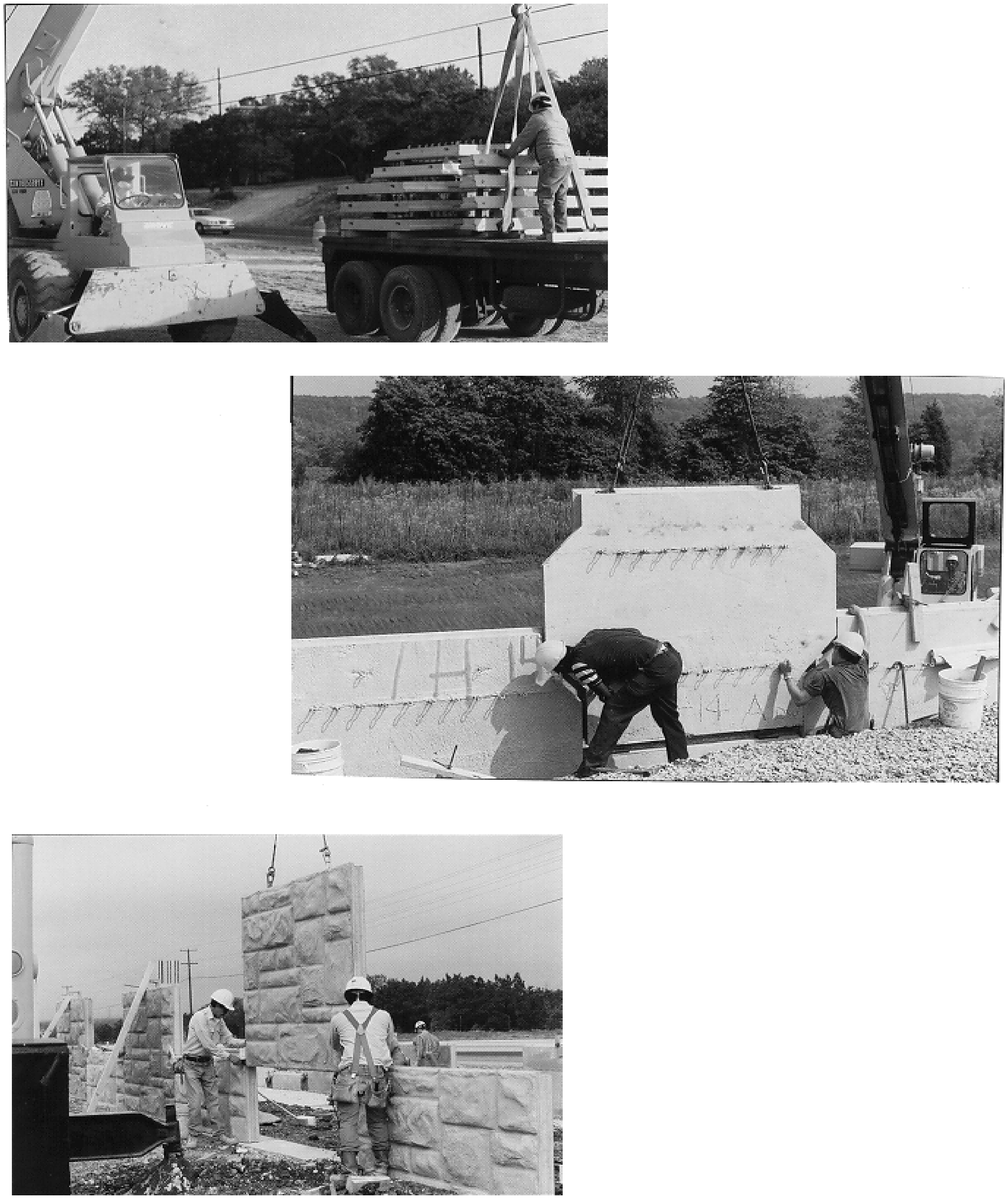


Figure 7. Erection of Precast Panels.

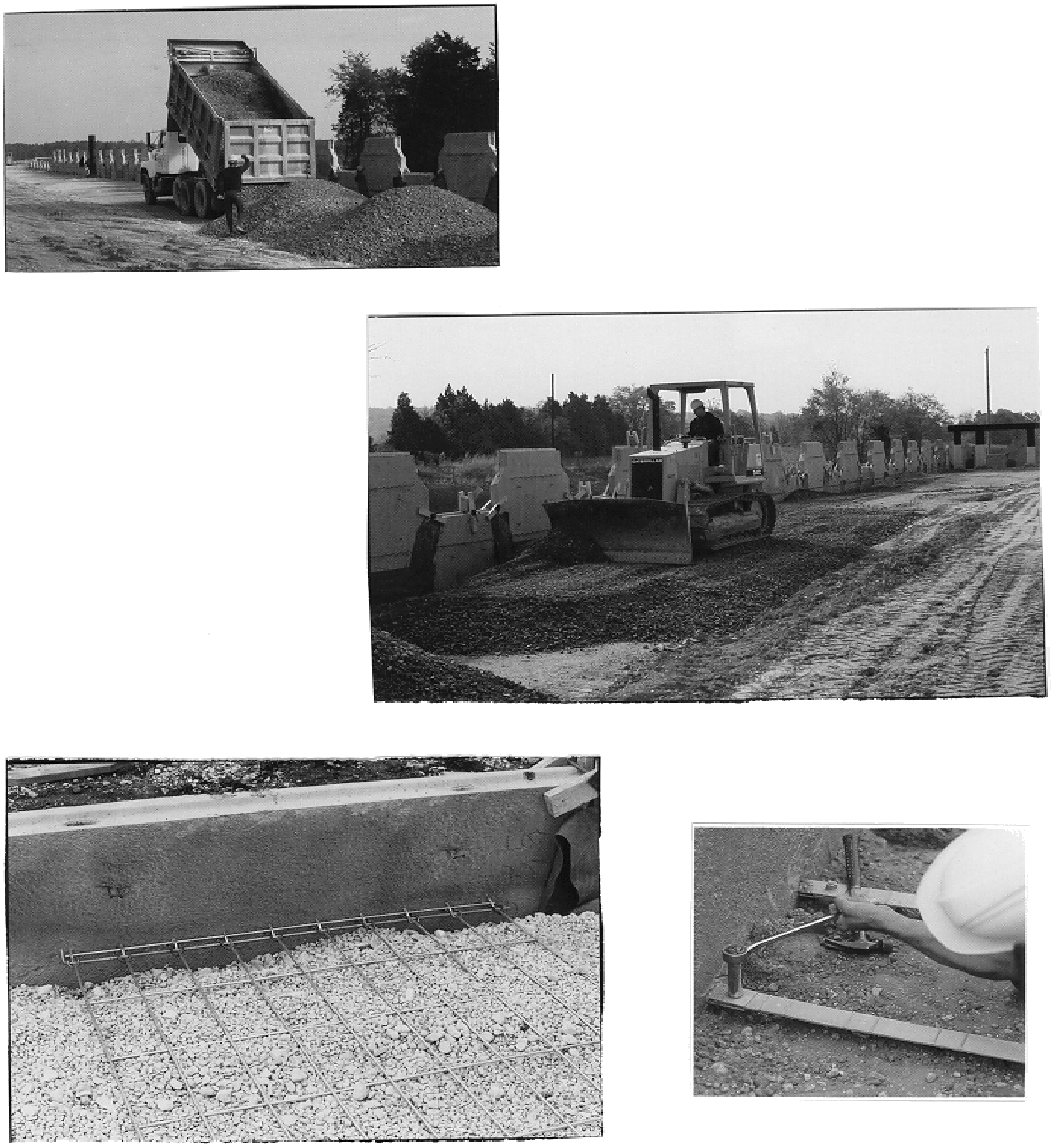


Figure 8. Fill Spreading and Reinforcement Connection.

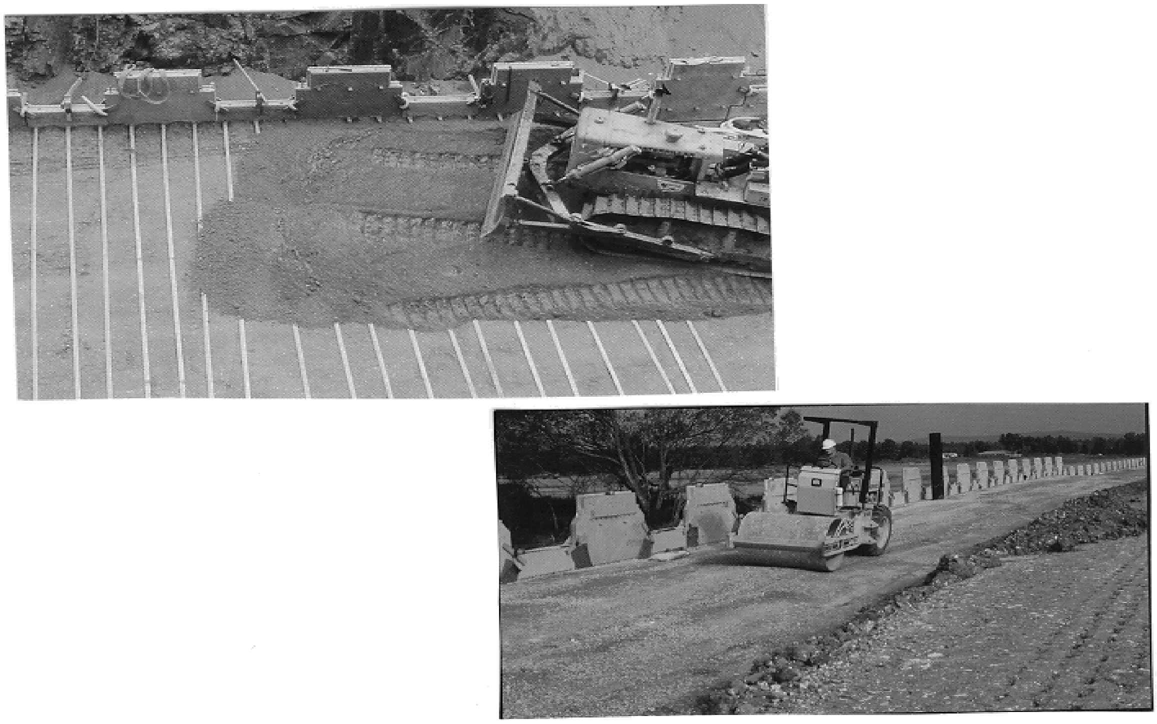


Figure 9. Compaction of Backfill.

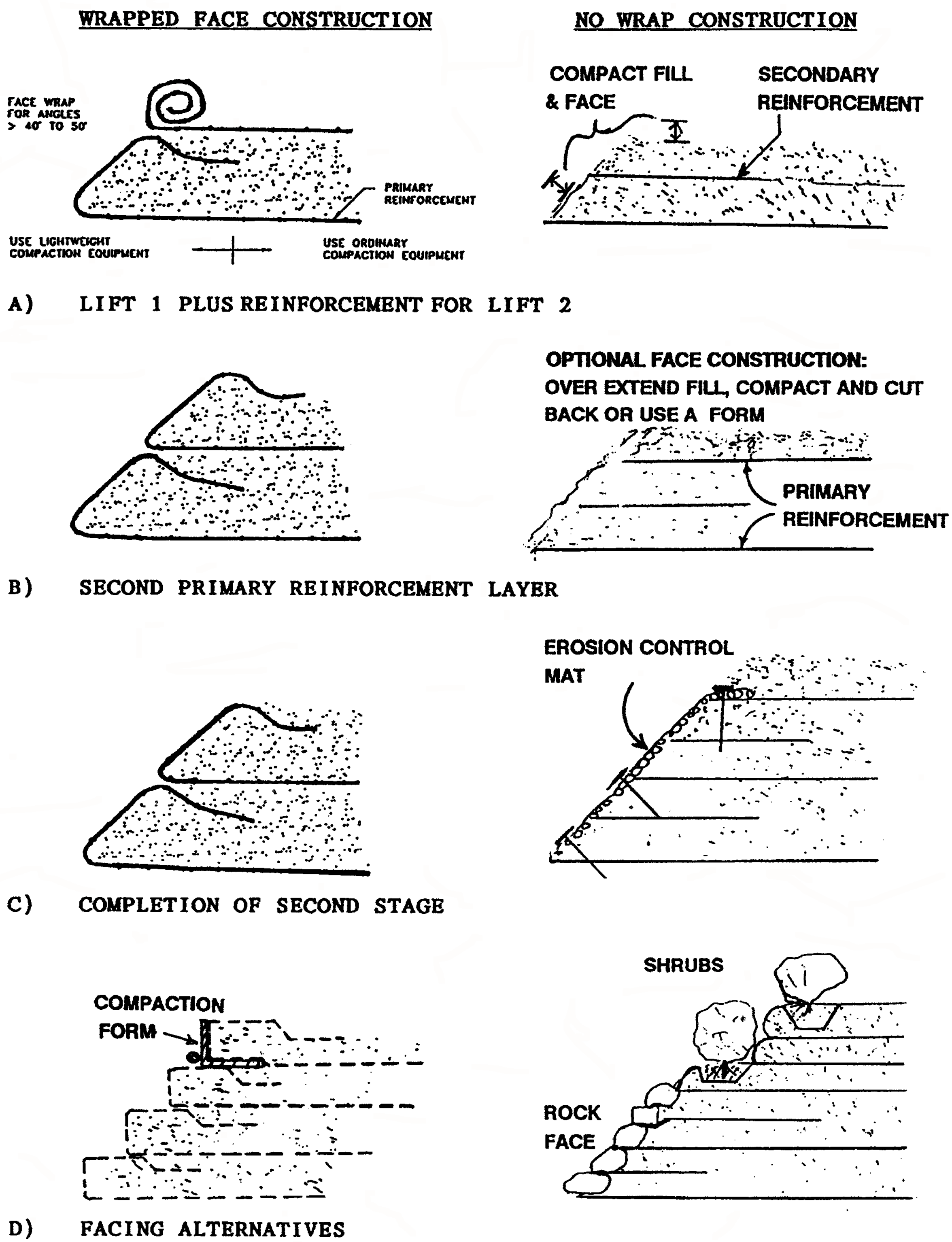


Figure 10. Construction of Reinforced Soil Slopes.

CHAPTER 4
DESIGN OF MSE AND RSS SYSTEMS

4.1 REINFORCED SOIL CONCEPTS

A reinforced soil mass is somewhat analogous to reinforced concrete in that the mechanical properties of the mass are improved by reinforcement placed parallel to the principal strain direction to compensate for the soil's lack of tensile resistance. The improved tensile properties are a result of the interaction between the reinforcement and the soil. The composite material has the following characteristics:

- Stress transfer between the soil and reinforcement takes place continuously along the reinforcement.
- Reinforcements are distributed throughout the soil mass with a degree of regularity and must not be localized.

Stress Transfer Mechanisms

Stresses are transferred between soil and reinforcement by friction and/or passive resistance depending on reinforcement geometry.

Friction develops at locations where there is a relative shear displacement and corresponding shear stress between soil and reinforcement surface. Reinforcing elements where friction is important should be aligned with the direction of soil reinforcement relative movement. Examples of such reinforcing elements are steel strips, longitudinal bars in grids, geotextile, and some geogrid layers.

Passive resistance occurs through the development of bearing type stresses on "transverse" reinforcement surfaces normal to the direction of soil reinforcement relative movement. Passive resistance is generally considered to be the primary interaction for rigid geogrids, bar mat, and wire mesh reinforcements. The transverse ridges on "ribbed" strip reinforcement also provide some passive resistance.

The contribution of each transfer mechanism for a particular reinforcement will depend on the roughness of the surface (skin friction), normal effective stress, grid opening dimensions, thickness of the transverse members, and elongation characteristics of the reinforcement. Equally important for interaction development are the soil characteristics, including grain size, grain size distribution, particle shape, density, water content, cohesion, and stiffness.

The primary function of reinforcements is to restrain soil deformations. In so doing, stresses are transferred from the soil to the reinforcement. These stresses are carried by the reinforcement in two ways: in tension or in shear and bending.

4.2 SOIL REINFORCEMENT INTERACTION

The design of the soil reinforcement system requires an evaluation of the long-term pullout performance with respect to three basic criteria:

- **Pullout capacity**, i.e., the pullout resistance of each reinforcement should be adequate to resist the design working tensile force in the reinforcement with a specified factor of safety.

- **Allowable displacement**, i.e., the relative soil-to-reinforcement displacement required to mobilize the design tensile force should be smaller than the allowable displacement.

- **Long-term displacement**, i.e., the pullout load should be smaller than the critical creep load.

The pullout resistance of the reinforcement is mobilized through one or a combination of the two basic soil-reinforcement interaction mechanisms, i.e., interface friction and passive soil resistance against transverse elements of composite reinforcements, such as bar mats, wire meshes, or geogrids. The load transfer mechanisms mobilized by a specific reinforcement depends primarily upon its structural geometry (i.e., composite reinforcement, such as grids versus linear or planar elements, thickness of transverse elements, and aperture dimension). The soil-to-reinforcement relative movement required to mobilize the design tensile force depends mainly upon the load transfer mechanism, the extensibility of the reinforcement material, the soil type, and the confining pressure.

The pullout resistance of the reinforcement is defined by the ultimate tensile load required to generate outward sliding of the reinforcement through the reinforced soil mass. Several approaches and design equations have been developed and are currently used to estimate the pullout resistance by considering frictional resistance, passive resistance, or a combination of both. Herein, a unified normalized approach presented by Elias et al. (2001) is adopted.

The pullout resistance, P_r, of the reinforcement per unit width of reinforcement is given by:

$$P_r = F^* . \alpha . \sigma_{vo}' . L_e . C \qquad (6\text{-}1)$$

where:

$L_e \cdot C$	=	the total surface area per unit width of the reinforcement in the resistive zone behind the failure surface
L_e	=	the embedment or adherence length in the resisting zone behind the failure surface
C	=	the reinforcement effective unit perimeter; e.g., C = 2 for strips, grids, and sheets
F^*	=	the pullout resistance (or friction-bearing-interaction) factor
α	=	a scale effect correction factor to account for a non linear stress reduction over the embedded length of highly extensible reinforcements, based on laboratory data (generally 1.0 for metallic reinforcements and 0.6 to 1.0 for geosynthetic reinforcements).
σ'_v	=	the effective vertical stress at the soil-reinforcement interfaces.

The correction factor α depends, therefore, primarily upon the strain softening of the compacted granular backfill material, the extensibility and the length of the reinforcement. For inextensible reinforcement, α is approximately 1, but it can be substantially smaller than 1 for extensible reinforcements. The α factor (a scale correction factor) can be obtained from pullout tests on reinforcements with different lengths as presented in appendix A or derived using analytical or numerical load transfer models which have been "calibrated" through numerical test simulations. In the absence of test data, $\alpha = 0.8$ for geogrids and $\alpha = 0.6$ for geotextiles (extensible sheets) is recommended (Elias et al., 2001).

In absence of site-specific pullout testing data, it is reasonable to use these semi-empirical relationships in conjunction with the standard specifications for backfill to provide a conservative evaluation of pullout resistance.

For steel ribbed reinforcement, the Pullout Resistance Factor F* is commonly taken as

$F^* = \tan \rho = 1.2 + \log C_u$ at the top of the structure = 2.0 maximum

$F^* = \tan \phi$ at a depth of 6 m (20 ft.) and below

where

ρ is the interface friction angle mobilized along the reinforcement, ϕ is the wall fill peak friction angle, and C_u is the uniformity coefficient of the backfill (D_{60}/D_{10}). **If**

the specific C_u for the wall backfill is unknown at design time, a C_u of 4 should be assumed (i.e., F* = 1.8 at the top of the wall), for backfills meeting the requirements of Chapter 3.

For steel grid reinforcements with transverse spacing $S_t \geq 150$ mm (6 in.), F* is a function of a bearing or embedment factor (F_q), applied over the contributing bearing α_β, as follows:

$$F^* = F_q\,\alpha_\beta = 40\,\alpha_\beta = 40\,(t/2S_t) = 20\,(t/S_t) \text{ at the top of the structure}$$
$$F^* = F_q\,\alpha_\beta = 20\,\alpha_\beta = 20\,(t/2S_t) = 10\,(t/S_t) \text{ at a depth of 6 m (20 ft.) and below}$$

where t is the thickness of the transverse bar. S_t shall be uniform throughout the length of the reinforcement, rather than having transverse grid members concentrated only in the resistant zone.

For geosynthetic (i.e., geogrid and geotextile) sheet reinforcement, the pullout resistance is based on a reduction in the available soil friction with the reduction factor often referred to as an Interaction Factor, C_i. In the absence of test data, the F* value for geosynthetic reinforcement should conservatively be taken as

$$F^* = 2/3 \tan \phi$$

Where used in the above relationships, ϕ is the peak friction angle of the soil which for MSE walls using select granular backfill, is taken as 34 degrees unless project specific test data substantiates higher values. For RSS structures, the ϕ angle of the reinforced backfill is normally established by test, as a reasonably wide range of backfills can be used. A lower bound value of 28 degrees is often used.

4.3 REINFORCEMENT TENSILE STRENGTH

Steel Reinforcement

For steel reinforcements, the design life is achieved by reducing the cross-sectional area of the reinforcement used in design calculations by the anticipated corrosion losses over the design life period as follows:

$$E_c = E_n - E_R$$

where E_c is the thickness of the reinforcement at the end of the design life, E_n the nominal thickness at construction, and E_R the sacrificial thickness of metal expected to be lost by uniform corrosion during the service life of the structure.

The strength of the reinforcement to resist the developed tension in the reinforcement is a function of reinforcement type. The allowable tensile force per unit width of reinforcement, T_a, is obtained as follows:

$$T_a = 0.55 \frac{F_y A_c}{b} \qquad \text{for steel strips}$$

and

$$T_a = 0.48 \frac{F_y A_c}{b} \qquad \text{for steel grids connected to concrete panels or blocks}$$

(Note: 0.55 F_y may be used for steel grids with flexible facings)

where:

- b = the gross width of the strip, sheet, or grid
- F_y = yield stress of steel
- A_c = design cross section area of the steel, defined as the original cross section area minus corrosion losses anticipated to occur during the design life of the wall

The corrosion rates presented below are suitable for conservative design. **These rates assume a mildly corrosive backfill material having the controlled electrochemical property limits that are discussed under electrochemical properties for reinforced fills.**

<u>Corrosion Rates</u> - mildly corrosive backfill

For zinc/side
15 µm/year (0.6 mils/yr.) (first 2 years)
4 µm/year (0.16 mils/yr.) (thereafter)

For residual carbon steel/side
12 µm/year (0.5 mils/yr.) (thereafter)

Based on these rates, complete corrosion of galvanization with the minimum required thickness of 86 µm (3.4 mils) (AASHTO M 111) is estimated to occur during the first 16 years and a carbon steel thickness or diameter loss of 1.42 mm to 2.02 mm (0.055 in to 0.08 in) would be anticipated over the remaining years of a 75 to 100 year design life, respectively. The designer of an MSE structure should also consider the potential for changes in the reinforced backfill environment during the structure's service life. In certain parts of the United States, it can be expected that deicing salts might cause such an environment change. For this problem, the depth of chloride infiltration and concentration are of concern.

For permanent structures directly supporting roadways exposed to deicing salts, limited data indicate that the upper 2.5 m (8 ft) of the reinforced backfill (as measured from the roadway surface) are affected by higher corrosion rates not presently defined. Under these conditions, it is recommended that a 30 mil (minimum) geomembrane be placed below the road base and tied into a drainage system to mitigate the penetration of the deicing salts in lieu of higher corrosion rates.

Geosynthetic Reinforcement

Selection of T_a for geosynthetic reinforcement is more complex than for steel. The tensile properties of geosynthetics are affected by environmental factors, such as creep, installation damage, aging, temperature, and confining stress. Furthermore, characteristics of geosynthetic products manufactured with the same base polymer can vary widely, and the details of polymer behavior for in-ground use are not completely understood. Ideally, T_a should be determined by thorough consideration of allowable elongation, creep potential, and all possible strength degradation mechanisms.

For geosynthetic reinforcements, the design life is achieved by developing an allowable design load, which considers all time-dependent strength losses over the design life period as follows

$$T_a = \frac{T_{ULT}}{RF \bullet FS} = \frac{T_{al}}{FS}$$

where T_a is the design long-term reinforcement tension load for the limit state, T_{ULT} is the ultimate geosynthetic tensile strength, RF is the product of all applicable reduction factors, and FS the overall factor of safety. T_{al} is the long-term material strength, or more specifically

$$T_{al} = \frac{T_{ULT}}{RF_{CR} \bullet RF_D \bullet RF_{ID}}$$

where:

T_{al} = Long-term tensile strength on a load per unit width of reinforcing basis.

T_{ULT} = Ultimate (or yield) tensile strength from wide strip test (ASTM D 4595) for geotextiles and wide strip (ASTM D 4595) or single rib test (GR1:GG1) for geogrids (*Note that the same test shall be used for definition of the geogrid creep reduction factor.*), based on minimum average roll value (MARV) for the product.

RF_{CR} = Creep Reduction Factor is the ratio of the ultimate strength (T_{ULT}) to the creep limit strength obtained from laboratory creep tests for each product.

RF_D = Durability reduction factor. It is dependent on the susceptibility of the geosynthetic to attack by microorganisms, chemicals, thermal oxidation, hydrolysis, and stress cracking, and can vary typically from 1.1 – 2.0. The minimum reduction factor shall be 1.1.

RF_{ID} = Installation Damage reduction factor. It can range from 1.05 – 3.0, depending on backfill gradation and product mass per unit weight. The minimum reduction factor shall be 1.1 to account for testing uncertainties.

FS = Overall factor of safety to account for uncertainties in the geometry of the structure, fill properties, reinforcement properties, and externally applied loads. For permanent, MSEW structures only, a minimum factor of safety of 1.5 has been typically used (thus $T_a = T_{al} / 1.5$).

For RSS structures, it is taken as 1.0, as the required factor of safety is accounted in the stability analysis (thus $T_a = T_{al}$).

T_{al} is typically obtained directly from the manufacturer. It typically includes reduction factors but does not include a design or material factor of safety, FS. The determination of reduction factors for each geosynthetic product require extensive field and/or laboratory testing, briefly summarized as follows:

Creep Reduction Factor, RF_{CR}

The creep reduction factor is obtained from long-term laboratory creep testing as detailed in FHWA NHI-00-043 (Elias et al., 2001). This reduction factor is required to limit the load in the reinforcement to a level known as the creep limit, that will preclude creep rupture over the life of the structure. Creep in itself does not degrade the strength of the polymer. Creep testing is essentially a constant load test on multiple product samples, loaded to various percentages of the ultimate product load, for periods of up to 10,000 hours. The creep reduction factor is the ratio of the ultimate load to the extrapolated maximum sustainable load (i.e., creep limit) within the design life of the structure (e.g., several years for temporary structures, 75 to 100 years for permanent structures). Typical reduction factors as a function of polymer type are indicated in Table 6.

Table 6. Creep reduction factors, RF_{CR}.

Polymer Type	Creep Reduction Factors, RF_{CR}
Polyester	2.5 to 1.6
Polypropylene	5 to 4.0
Polyethylene	5 to 2.6

Durability Reduction Factor, RF_D

With respect to aging degradation, current research results suggest the following:

Polyester Geosynthetics

PET geosynthetics are recommended for use in environments characterized by $3 < pH < 9$, only. The reduction factors for PET aging (RF_D) listed in Table 7 are presently indicated for a 100-year design life in the absence of product specific testing.

Table 7. Aging reduction factors, PET.

No.	**Product***	**Reduction factor, RF_D**	
		$5 \leq pH \leq 8$	$3 \leq pH \leq 5$ $8 \leq pH \leq 9$
1	Geotextiles M_n<20,000, 40 <CEG<50	1.6	2.0
2	Coated geogrids, Geotextiles M_n>25,000, CEG<30	1.15	1.3

M_n = number average molecular weight CEG = carboxyl end group

* Use of materials outside the indicated pH or molecular property range requires specific product testing.

Polyolefin Geosynthetics

Since each product has a unique and proprietary blend of antioxidants, product specific testing is required to determine the effective life span of protection at the in-ground oxygen content. Limited data suggests that certain antioxidants are effective for up to 100 years in maintaining strength for in-ground use.

Installation Damage Reduction Factor, RF_{ID}

The placing and compaction of the backfill material against the geosynthetic reinforcement may reduce its tensile strength. The level of damage for each geosynthetic reinforcement is a variable and is a function of the weight and type of the construction equipment and the type of geosynthetic material. The installation damage is also influenced by the lift thickness and type of soil present on either side of the reinforcement. Where granular and angular soils are used for backfill, the damage is more severe than where softer, finer, soils are used.

The following recommendations are stated in this companion document in regards to defining a RF_{ID} factor. For more detailed explanations, see *FHWA-NHI-00-044* "Corrosion/Degradation of Soil Reinforcements for Mechanically Stabilized Earth Walls and Reinforced Soil Slopes," (Elias, 2000). To account for installation damage losses of strength where full-scale product-specific testing is not available, Table 8 may be used with consideration of the project specified backfill characteristics. In absence of project specific data the largest indicated reduction factor for each geosynthetic type should be used.

Table 8. Installation damage reduction factors (Elias, 2000).

Reduction Factor, RF_{ID}			
No.	**Geosynthetic**	**Type 1 Backfill Max. Size 102mm D_{50} about 30mm**	**Type 2 Backfill Max. Size 20mm D_{50} about 0.7mm**
1	HDPE uniaxial geogrid	1.20 - 1.45	1.10 - 1.20
2	PP biaxial geogrid	1.20 - 1.45	1.10 - 1.20
3	PVC coated PET geogrid	1.30 - 1.85	1.10 - 1.30
4	Acrylic coated PET geogrid	1.30 - 2.05	1.20 - 1.40
5	Woven geotextiles (PP&PET)[(1)]	1.40 - 2.20	1.10 - 1.40
6	Non woven geotextiles (PP&PET)[(1)]	1.40 - 2.50	1.10 - 1.40
7	Slit film woven PP geotextile [(1)]	1.60 - 3.00	1.10 - 2.00

(1) Minimum weight 270 g/m2 (7.9 oz/yd2)

Factor of Safety, FS

This is a global factor of safety which accounts for uncertainties in externally applied loads, structure geometry, fill properties, potential for local overstress due to load nonuniformity and uncertainties in long-term reinforcement strength. For ultimate limit state conditions, a FS of 1.5 for permanent walls and 1.2 for temporary walls is recommended (Elias et al., 2001, AASHTO 1997).

4.4 DESIGN METHODS FOR MSE WALLS

Since the development of soil reinforcement concepts and their application to MSEW structure design, a number of design methods have been proposed, used, and refined. Current practice consists of determining the geometric and reinforcement requirements to prevent internal and external failure using limit equilibrium methods of analysis.

Limit Equilibrium Analysis

A limit equilibrium analysis consists of a check of the overall stability of the structure. The types of stability that must be considered are external, internal, and combined:

- External stability involves the overall stability of the stabilized soil mass considered as a whole and is evaluated using slip surfaces outside the stabilized soil mass.
- Internal stability analysis consists of evaluating potential slip surfaces within the reinforced soil mass.
- A combined external/internal stability analysis may be required when the critical slip surface is partially outside and partially inside the stabilized soil mass.

a. Design Methods, Inextensible Reinforcements

The current method of limit equilibrium analysis uses a coherent gravity structure approach to determine external stability of the whole reinforced mass, and is similar to the analysis for any conventional or traditional gravity structure. For internal stability evaluations, it considers a bi-linear critical slip surface that divides the reinforced mass in active and resistant zones and requires that an equilibrium state be achieved for successful design.

The state of stress for **external stability** is assumed to be equivalent to a Coulomb state of stress with a wall friction angle δ equal to zero. For **internal stability,** a variable state of stress varying from a multiple of K_a to an active earth pressure state, K_a, is used in design.

Recent research (FHWA RD 89-043) has focused on developing the state of stress for internal stability, as a function of K_a, type of reinforcement used (geotextile, geogrid, metal strip, or metal grid), and depth from the surface. The results from these efforts have been synthesized in a *simplified coherent gravity method* that represents the state-of-the-practice.

b. Design Methods, Extensible Reinforcements

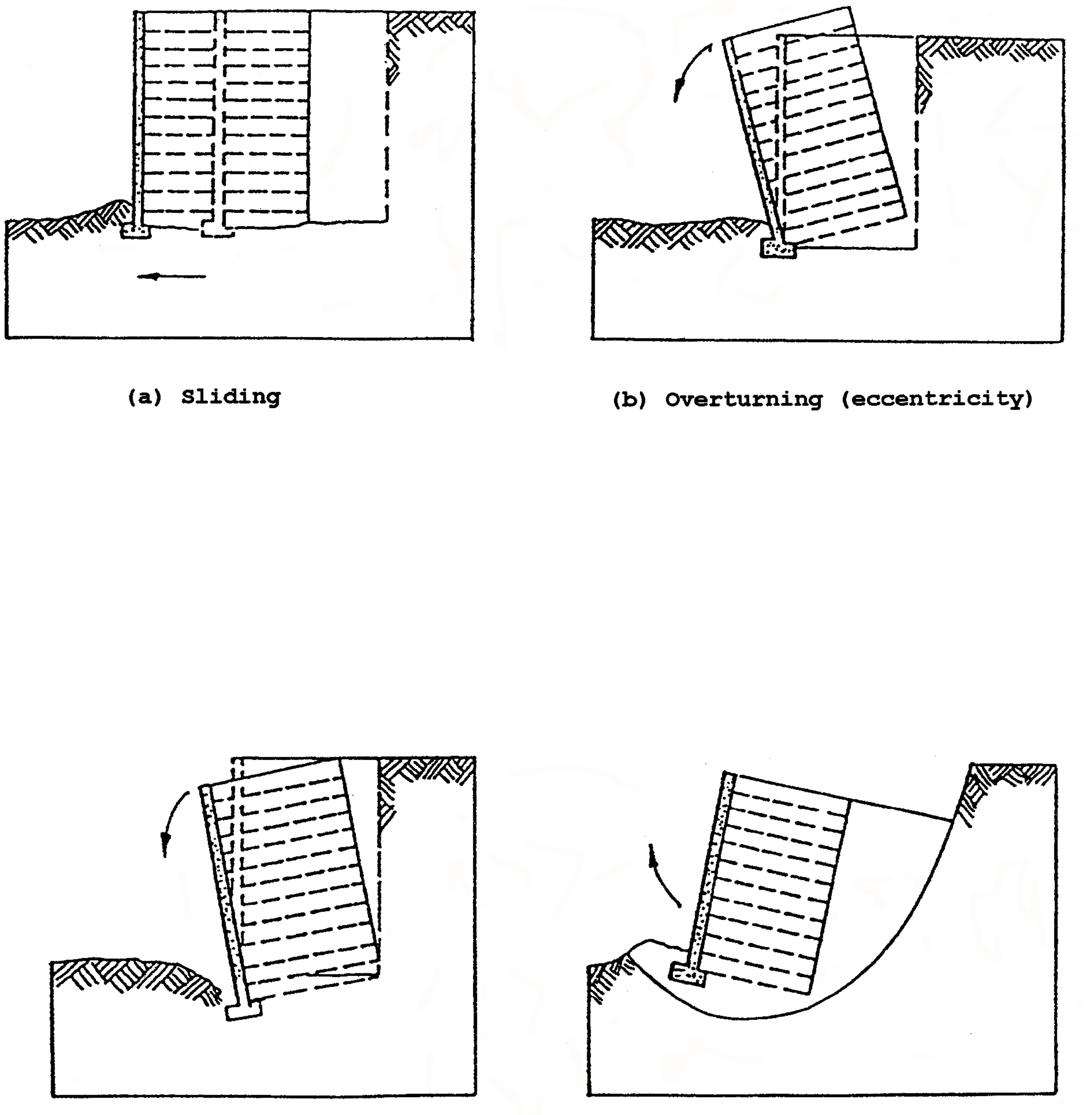
For **external stability** calculations, the current method assumes an earth pressure distribution consistent with the method used for inextensible reinforcements.

For **internal stability** computations using the *simplified coherent gravity method*, the internal coefficient of earth pressure is again a function of the type of reinforcement, where the minimum coefficient (K_a) is used for walls constructed with continuous sheets of geotextiles and geogrids. For internal stability, a Rankine failure surface is considered because the extensible reinforcements can elongate more than the soil before failure.

4.5 SIZING FOR EXTERNAL STABILITY

As with classical gravity and semigravity retaining structures, four potential external failure mechanisms are usually considered in sizing MSE walls, as shown in Figure 11. They include

- sliding on the base
- limiting the location of the resultant of all forces (overturning)
- bearing capacity
- deep-seated stability (rotational slip-surface or slip along a plane of weakness)



(a) Sliding

(b) Overturning (eccentricity)

(c) Bearing capacity

(d) Deep seated stability (Rotational)

Figure 11. Potential External Failure Mechanisms for an MSE Wall.

Due to the flexibility and satisfactory field performance of MSE walls, the adopted values for the factors of safety for external failure are in some cases lower than those used for reinforced concrete cantilever or gravity walls. For example, the factor of safety for overall bearing capacity may be 2.5, rather than the conventional value, which is used for more rigid structures.

Likewise, the flexibility of MSE walls should make the potential for overturning failure highly unlikely. However, overturning criteria (maximum permissible eccentricity) aid in controlling lateral deformation by limiting tilting and, as such, should always be satisfied.

External stability computations are sequentially performed as follows:

1) Preliminary sizing. Choose a trial reinforcement length of at least 70% of the height, but at least 2.5 m (8.2 ft.) in length.

2) Compute the earth pressure coefficient using a Coulomb state of stress. For a vertical wall and a horizontal backfill, the earth pressure coefficient is

$$K_a = \tan^2(45 - \phi/2) \tag{3}$$

where ϕ is the friction angle of the retained fill.

3) Compute vertical and horizontal forces acting on the reinforced zone, as shown in Figure 12.

4) Check that the preliminary sizing with respect to sliding along the base is adequate, or

$$FS_{sliding} = \frac{\sum horizontal\ resisting\ forces}{\sum horizontal\ sliding\ forces} \geq 1.5 \tag{4}$$

5) Compute the vertical stress at the foundation level by calculating first the eccentricity of all resultant forces and then the vertical stress on a reduced base width, as shown in Figure 13.

The eccentricity of all loads at the base must be

$e \leq L/6$ in soil
$e \leq L/4$ in rock

6) Check that the vertical stress does not exceed the allowable bearing capacity, or

$$\sigma_v \leq q_a = \frac{q_{ult}}{FS} = \frac{q_{ult}}{2.5} \tag{5}$$

where any of the above conditions are not satisfied, an increase in the length of reinforcements is indicated.

Additionally, overall and compound stability and foundation settlements must be considered when sizing for external stability. Overall stability is determined using rotational or wedge analyses, as appropriate, which can be performed using a classical slope stability analysis method. For simple structures with rectangular geometry, relatively uniform reinforcement spacing, and a near vertical face, compound failures passing both through the unreinforced and reinforced zones will not generally be critical. **However, if complex conditions exist, such as changes in reinforced soil types or reinforcement lengths, high surcharge loads, sloping faced structures, significant slopes at the toe or above the wall, or stacked structures, compound failures must be considered.** Refer to FHWA-NHI-00-043 for overall stability, compound stability, and settlement analysis techniques; and for sloping surcharge conditions, concentrated loads, seismic loads, or other unusual loading or geometric conditions.

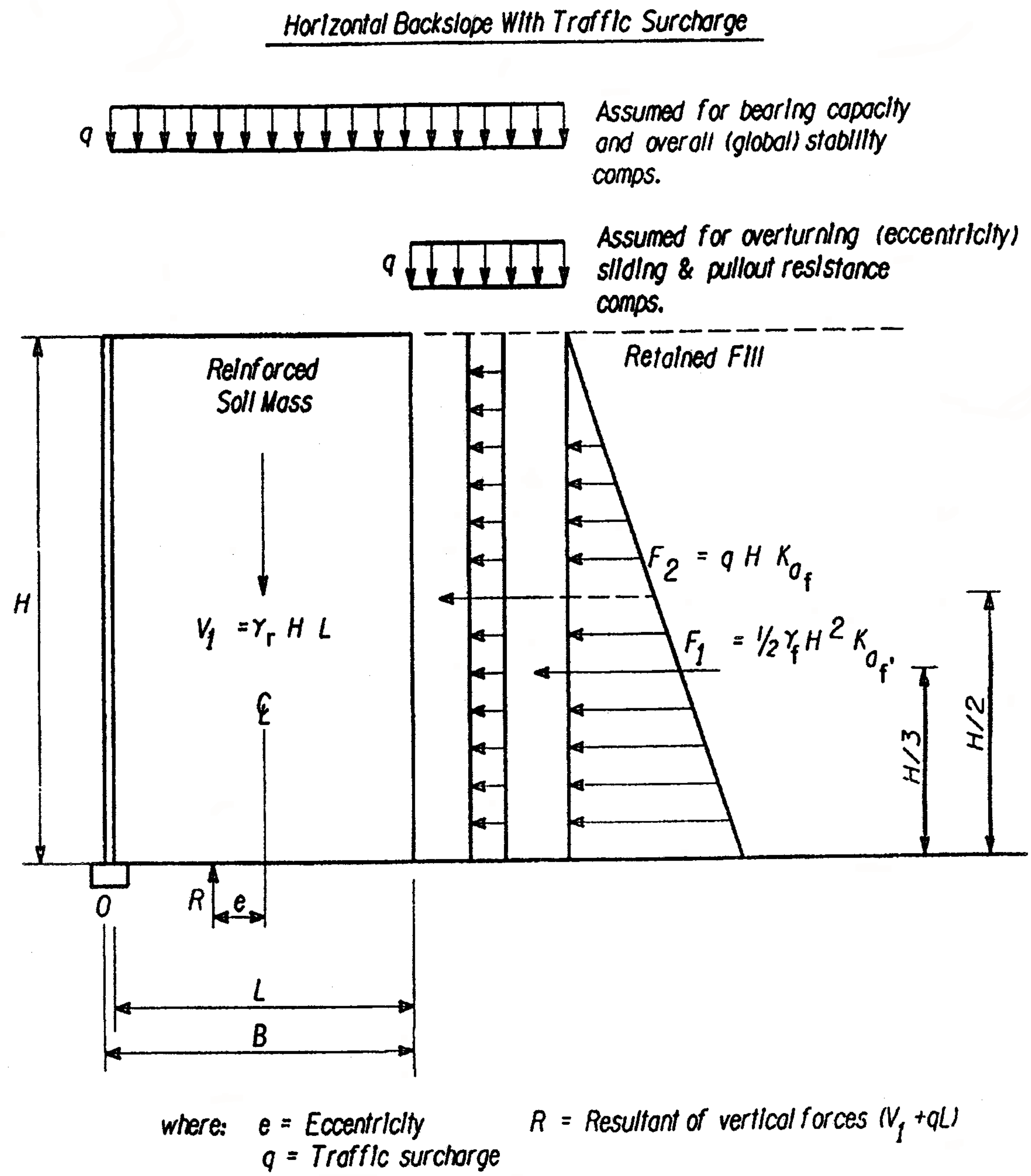
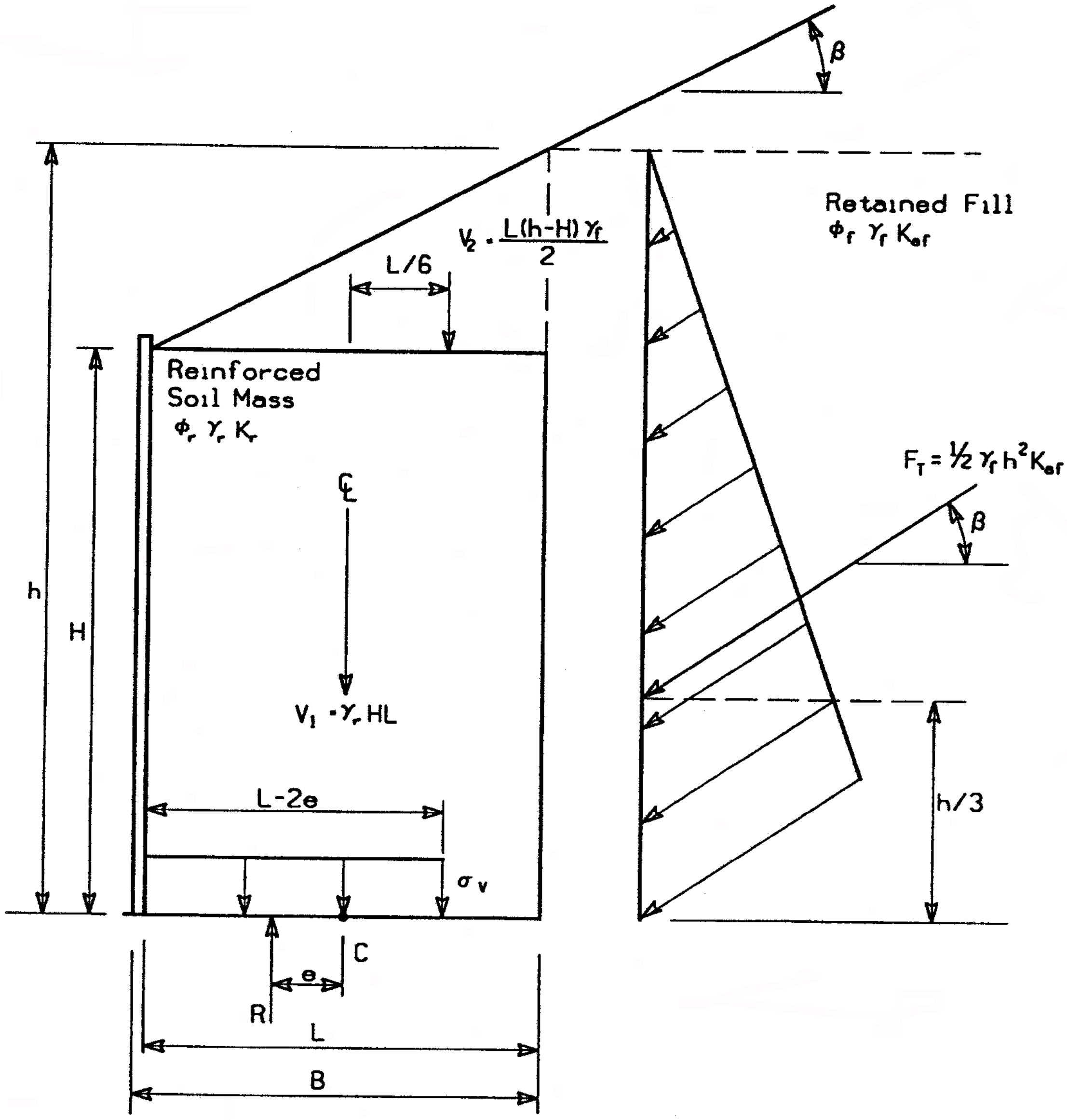


Figure 12. External Analysis: Earth Pressures/Eccentricity.
Horizontal Backslope with Traffic Surcharge.



R = Resultant of vertical forces

Note: For relatively thick facing elements (e.g., segmental concrete facing blocks) it may be desirable to include the facing dimensions and weight in bearing capacity calculations (i.e., use "B" in lieu of "L").

Figure 13. Calculation of Vertical Stress for Bearing Capacity (Sloping Backslope Condition).

4.6 SIZING FOR INTERNAL STABILITY

The process of sizing and designing to preclude internal failure consists of determining the maximum developed tension forces in the reinforcements, their location along a locus of critical slip surfaces, and the resistance provided by the reinforcements both in pullout capacity and tensile strength.

a. Critical Slip Surfaces

The most critical slip surface in a simple reinforced soil wall is assumed to coincide with the maximum tensile forces line (i.e., the locus of the maximum tensile force, T_{max}, in each reinforcement layer). The shape and location of this line is assumed to be known for simple structures from a large number of previous experiments and theoretical studies.

This maximum tensile forces surface has been assumed to be approximately bilinear in the case of inextensible reinforcements (Figure 14a), approximately linear in the case of extensible reinforcements (Figure 14b), and passes through the toe of the wall in both cases.

b. Calculation of Maximum Tensile Forces in the Reinforcement Layers

Recent research studies have indicated that the maximum tensile force is primarily related to the type of reinforcement in the MSE mass, which, in turn, is a function of the modulus, extensibility, and density of reinforcement. Based on this research, a relationship between the type of the reinforcement and the overburden stress has been developed and is shown in Figure 15. The resulting K/Ka for inextensible reinforcement ratio decreases from the top of wall to a constant value below 6 m (20 ft.).

The simplified approach used herein was developed in order to avoid iterative design procedures required by some of the complex refinements of the available methods i.e., the coherent gravity method (AASHTO, 1994 Interims) and the structure stiffness method (FHWA RD 89-043). The *simplified coherent gravity* method is based on the same empirical data used to develop these two methods.

This graphical figure was prepared by back analysis of the lateral stress ratio, K, from available field data, where stresses in the reinforcements have been measured and normalized as a function of an active earth pressure coefficient, K_a. The lines shown on the figure correspond to usual values representative of the specific reinforcement systems that are known to give satisfactory results, assuming that the vertical stress is equal to the weight of the overburden (γH). This provides a simplified evaluation method for all cohesionless reinforced fill walls. Future data may lead to modifications in Figure 15, including relationships for newly developed reinforcement types, effect of full-height panels, etc.

The lateral earth pressure coefficient, K, is determined by applying a multiplier to the active earth pressure coefficient. The active earth pressure coefficient is determined using a Coulomb earth pressure relationship, assuming no wall friction and a β angle equal to zero. For a vertical wall the earth pressure, therefore, reduces to the Rankine equation:

$$K_a = \tan^2 (45 - \phi'/2) \tag{6}$$

For wall face batters equal to or greater than 8 degrees from the vertical, the following simplified form of the Coulomb equation can be used:

$$K_a = \frac{\sin^2(\theta + \phi')}{\sin^3 \theta \left[1 + \frac{\sin \phi'}{\sin \theta}\right]^2} \tag{7}$$

where θ is the inclination of the back of the facing as measured from the horizontal starting in front of the wall.

The vertical stress (γH) is the result of gravity forces from soil self weight within and immediately above the reinforced wall backfill, and any surcharge loads present.

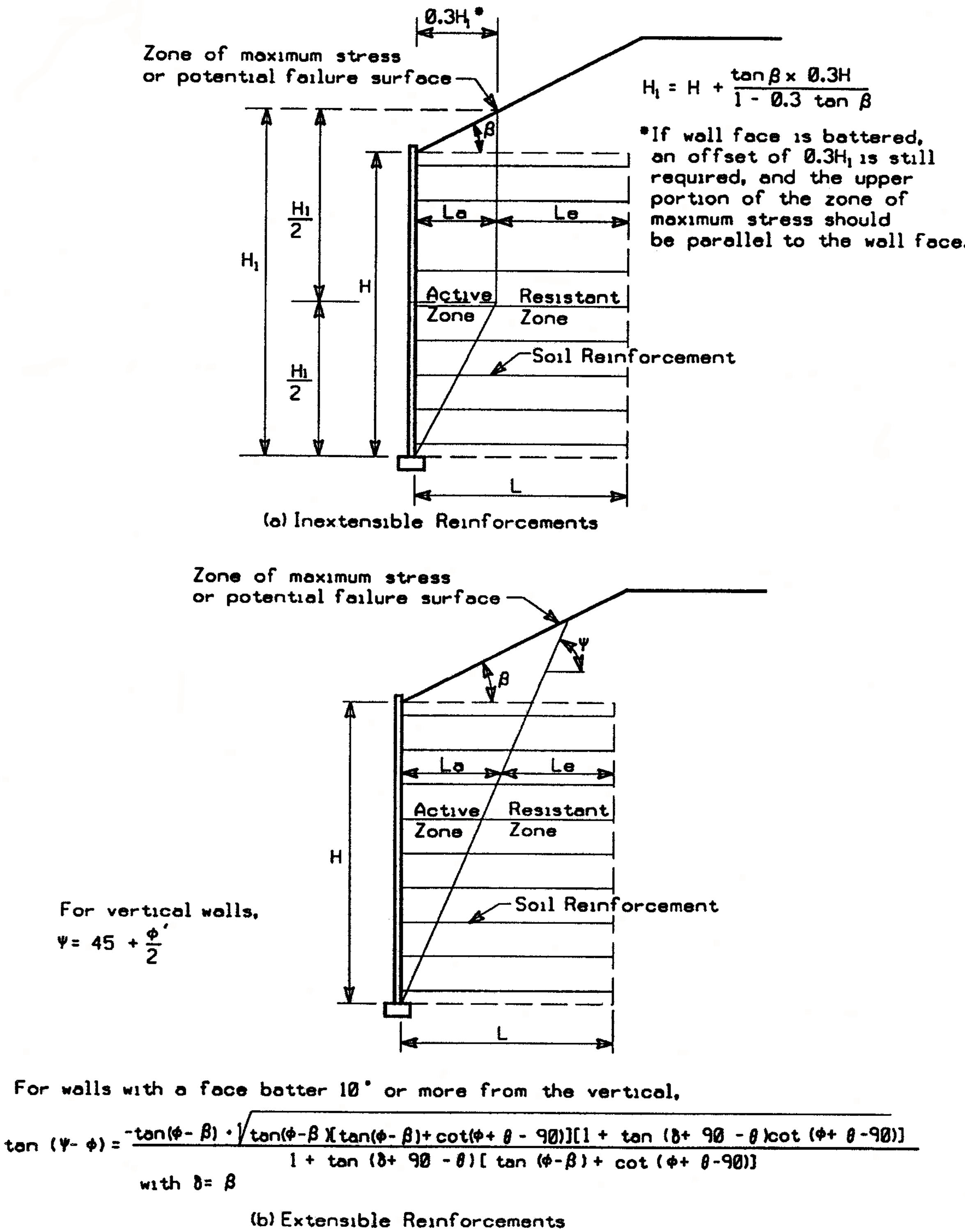


Figure 14. Location of Potential Failure Surface for Internal Stability Design of MSE Walls.

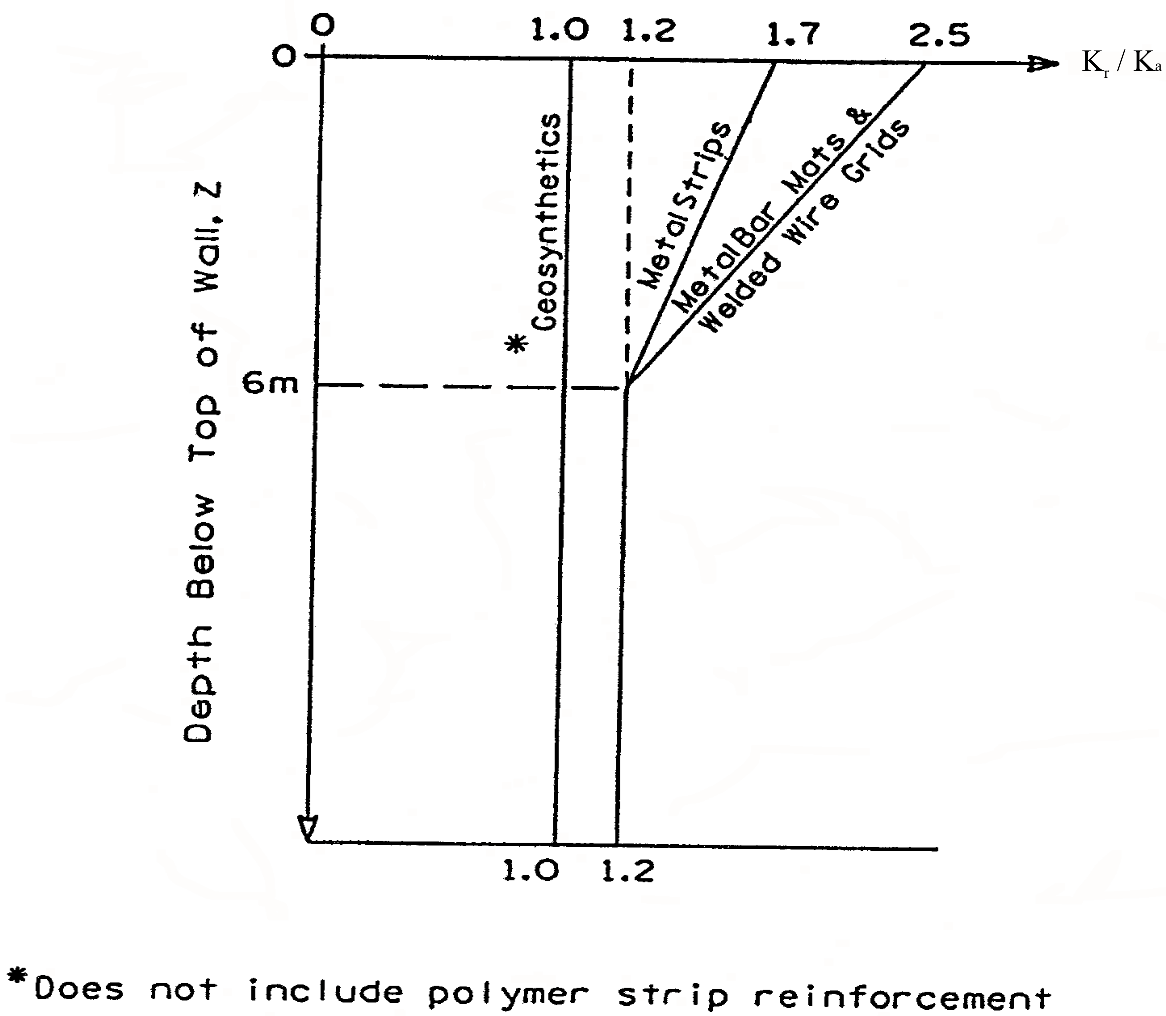


Figure 15. Variation of the Stress Ratio with Depth in an MSE Wall.

Calculations steps are as follows:

(1) Calculate at each reinforcement level, the horizontal stresses, σ_h, along the potential failure line from the weight of the retained fill $\gamma_r Z$ plus, if present, uniform surcharge loads q, concentrated surcharge loads $\Delta\sigma_v$, and $\Delta\sigma_h$.

$$\sigma_H = K_r\,\sigma_v + \Delta\sigma_h \tag{8}$$

where:

$$\sigma_v = \gamma_r\,Z + \sigma_2 + q + \Delta\sigma_v$$

where: $K_r = K(z)$ is shown in Figure 15, and Z is the depth referenced below the top of wall, excluding any copings and appurtenances.

For sloping soil surfaces above the MSE wall section, the actual surcharge is replaced by a uniform surcharge, σ_v, equal to 0.5 γh_s, where h_s is the height of the slope at the back of the reinforcements.

(2) Calculate the maximum tension, T_{max}, in each reinforcement layer per unit width of wall based on the vertical spacing, S_v, from

$$T_{\max} = \sigma_H \bullet S_v \tag{9a}$$

T_{max} may be also be calculated at each level for discrete reinforcements (metal strips, bar mats, geogrids, etc.) per a defined unit width of reinforcement, from

$$T_{\max-R} = \frac{\sigma_H \bullet S_v}{R_c} \tag{9b}$$

where R_c is the coverage ratio b/S_h, with b the gross width of the reinforcing element, and S_h the center-to-center horizontal spacing between reinforcements (e.g., $R_c = 1$ for full coverage reinforcement).

Alternatively, for discrete reinforcements and segmental precast concrete facing, T_{max} is often more conveniently calculated per tributary area, A_t, defined as the area equal to the two (2) panel widths times the vertical spacing s_v.

$$T_{max-D} = \sigma_H \bullet A_t \quad (9c)$$

(3) Calculate internal stability with respect to breakage of the reinforcement. Stability with respect to breakage of the reinforcements requires that

$$T_a \geq T_{max} \quad (10)$$

Where T_a is the allowable tension force per unit width of the reinforcement.

c. Internal Stability with Respect to Pullout Failure

Stability with respect to pullout of the reinforcements requires that the following criteria be satisfied:

$$T_{max} \leq \frac{1}{FS_{PO}} F^* \gamma \, Z_p \, L_e \, C \, R_c \, \alpha \quad (11)$$

where:

FS_{PO} = Safety factor against pullout ≥ 1.5.

T_{max} = Maximum reinforcement tension.

C = 2 for strip, grid, and sheet type reinforcement.

α = Scale correction factor.

F* = Pullout resistance factor.

R_c = Coverage ratio.

$\gamma \, Z_p$ = The overburden pressure, including distributed dead load surcharges, neglecting traffic loads.

L_e = The length of embedment in the resisting zone. Note that the boundary between the resisting and active zones may be modified by concentrated loadings.

d. Computer-Assisted Design

The repetitive nature of the computations required at each level of reinforcement lends itself to computer-assisted design. The computer program MSEW[6] developed under FHWA sponsorship analyzes and/or designs MSE walls using any type of metallic or geosynthetic reinforcement in conjunction with any type of facing (precast concrete, MBW, etc.). Version 1.0 has been designated exclusively for use by U.S. State Highway Agencies and by U.S. Federal agencies. Version 1.1 is available to the public through ADAMA Engineering (www.MSEW.com).

4.7 DESIGN OF REINFORCED STEEPENED SLOPES (RSS)

a. Design Approach

The design of reinforcement for safe, steep slopes requires a rigorous analysis. The design of reinforcement for this application is critical, as failure of the reinforcement would result in failure of the slope.

The overall design requirements for reinforced slopes are similar to those for unreinforced slopes. The factor of safety must be adequate for both the short-term and long-term conditions and for all possible modes of failure.

As illustrated in Figure 16, there are three failure modes for reinforced slopes:

- Internal, where the failure plane passes through the reinforcing elements.
- External, where the failure surface passes behind and underneath the reinforced mass.
- Compound, where the failure surface passes behind and through the reinforced soil mass.

In some cases, the calculated stability safety factor can be approximately equal in two or all three modes, if the reinforcement strengths, lengths, and vertical spacing are optimized.[7]

For steepened reinforced slopes (vertical inclination up to 70 degrees) and slope repair, design is based on modified versions of the classical limit equilibrium slope stability methods as shown in Figure 17:

- Circular or wedge-type potential failure surface is assumed.

- The relationship between driving and resisting forces or moments determines the slope factor of safety.

- Reinforcement layers intersecting the potential failure surface are assumed to increase the resisting force or moment, based on their tensile capacity and orientation. (Usually, the shear and bending strengths of stiff reinforcements are not taken into account.)

- The tensile capacity of a reinforcement layer is taken as the minimum of its allowable pullout resistance behind (or in front of) the potential failure surface and its long-term allowable design strength.

- $T_{S\text{-}MAX}$ the largest T_S calculated and establishes the total design tension.

- **Note: The minimum safety factor usually does not control the location of $T_{S\text{-}MAX}$; the most critical surface is the surface requiring the largest magnitude of reinforcement.**

Calculate the total reinforcement tension per unit width of slope, T_S, required to obtain the required factor of safety, FS_R, for each potential failure surface inside the critical zone in step 5 that extends through or below the toe of the slope using the following equation:

$$T_s = (FS_R - FS_U)\frac{M_D}{D} \tag{10}$$

where:

T_S = the sum of the required tensile force per unit width of reinforcement (considering rupture and pullout) in all reinforcement layers intersecting the failure surface.

M_D = driving moment about the center of the failure circle.

D = the moment arm of T_S about the center of failure circle.

D = radius of circle R for **continuous, sheet type extensible reinforcement** (i.e., assumed to act tangentially to the circle).

D = radius of circle R for **continuous, sheet type inextensible reinforcement** (e.g., wire mesh reinforcement) to account for normal stress increase on adjacent soil.

D = vertical distance, Y, to the centroid of T_S for **discrete element, strip type reinforcement**. Assume H/3 above slope base for preliminary calculations (i.e., assumed to act in a horizontal plane intersecting the failure surface at H/3 above the slope base).

FS_U = unreinforced slope safety factor.

FS_R = target minimum slope factor of safety, which is applied to both the soil and reinforcement.

As shown in Figure 16, a wide variety of potential failure surfaces must be considered, including deep-seated surfaces through or behind the reinforced zone. The critical slope stability factor of safety is taken from the unreinforced failure surface requiring the maximum reinforcement. This is the failure surface with the largest unbalanced driving moment to resisting moment, and not the surface with the minimum calculated unreinforced factor of safety. This failure surface is equivalent to the critical reinforced failure surface with the lowest factor of safety. Detailed design of reinforced slopes is performed by determining the factor of safety with successively modified reinforcement layouts until the target factor of safety is achieved.

There are several approaches available for the design of slope reinforcement. Most methods use conventional slope stability computer programs and the steps necessary to manually calculate the reinforcement requirements for almost any condition. Figure 17 shows the conventional rotational slip surface method used in the analysis. Fairly complex conditions can be accommodated depending on the analytical method used (e.g., Bishop, Janbu).

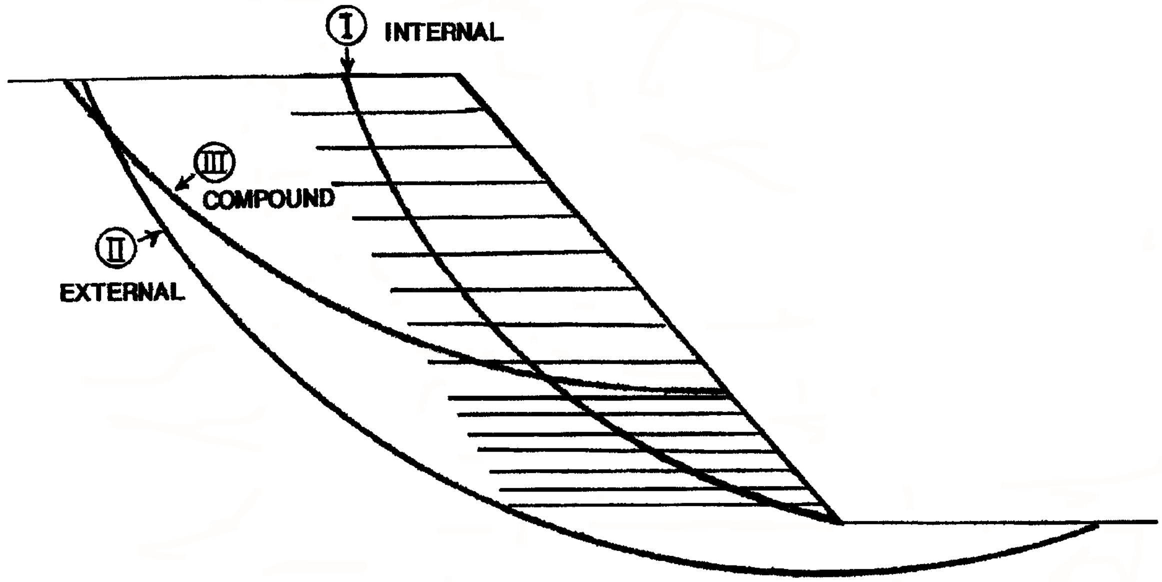
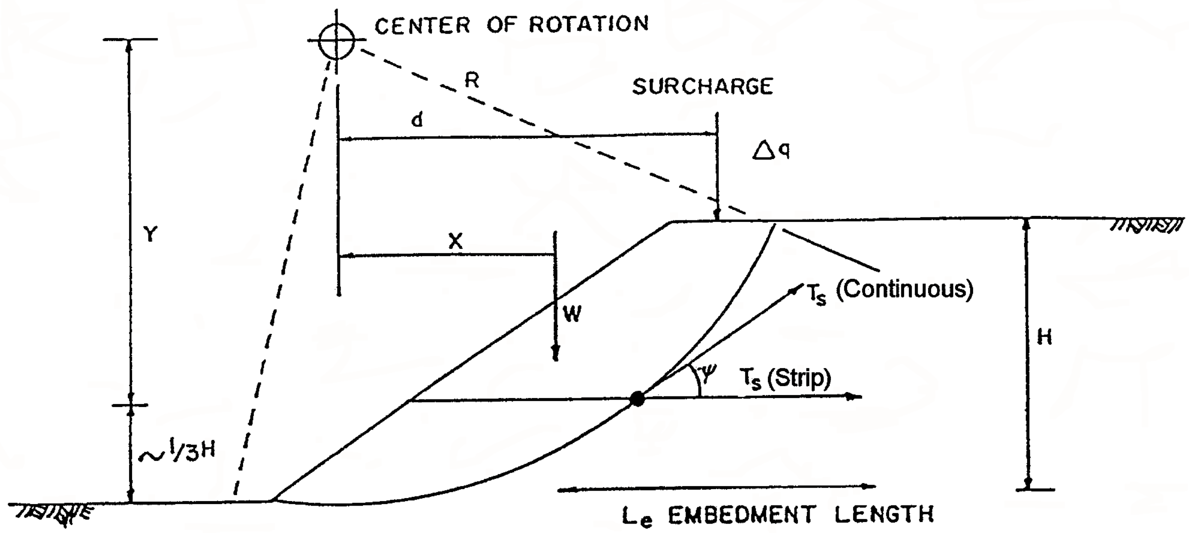


Figure 16. Failure Modes for Reinforced Soil Slopes.



Factor of safety of unreinforced slope:

$$F.S._u = \frac{\text{Resisting Moment } (M_R)}{\text{Driving Moment } (M_D)} = \frac{\int_0^{L_{sp}} \tau_f \cdot R \cdot dL}{(Wx + \Delta q \cdot d)}$$

where: W = weight of sliding earth mass
L_{sp} = length of slip plane
Δq = surcharge
τ_f = shear strength of soil

Factor of safety of reinforced slope:

$$F.S. = F.S._u + \frac{T_s \cdot D}{M_D}$$

where: T_s = sum of available tensile force per width of reinforcement for all reinforcement layers
D = moment arm of T_s about the center of rotation
= R for continuous extensible and inextensible reinforcement
= Y for discrete reinforcement

Figure 17. Rotational Shear Approach to Determine Required Strength of Reinforcement.

b. Computer-Assisted Design

The ideal method for reinforced slope design is to use a conventional slope stability computer program that has been modified to account for the stabilizing effect of reinforcement. Such programs should account for reinforcement strength and pullout capacity, compute reinforced and unreinforced safety factors automatically, and have some searching routine to help locate critical surfaces. The method would also include the confinement effects of the reinforcement on the shear strength of the soil in the vicinity of the reinforcement.

A generic, Windows™-based program ReSSA[8] developed under FHWA sponsorship analyzes and/or designs RSS structures using geosynthetic reinforcement. It follows FHWA-NHI-00-043, *Mechanically Stabilized Earth Walls and Reinforced Soil Slopes, Design and Construction Guidelines,* and portions of the manual are incorporated in the Help menu. ReSSA has two modes of operation: *Design* and *Analysis*. In the *Design* mode, the program computes the required layout (length and vertical spacing) corresponding to the user's prescribed safety factors. In this mode, the program produces the *ideal* reinforcement values for strength or coverage ratio so that the designer can maximize reinforcement utilization. In the *Analysis* mode, ReSSA computes the factors of safety corresponding to the user's prescribed layout.

Several other reinforced slope programs are commercially available. These programs generally do not design the reinforcement but allow for an evaluation of a given reinforcement layout. An iterative approach then follows to optimize either the reinforcement strength or layout. Most of the programs are limited to simple soil profiles and, in some cases, simple reinforcement layouts. Also, external stability evaluation is generally limited to specific soil and reinforcement conditions and a single mode of failure. In some cases, the programs are reinforcement specific.

c. Preliminary Feasibility Design

Preliminary design for feasibility evaluation can be easily made by the use of design charts. These charts could be used for final design of low walls for applications where the consequence of failure are non-critical. Figure 18 is a widely used chart presenting a simplified method, based on a two-part, wedge-type failure surface, whose use is limited by the assumptions noted on the figure. Note that Figure 18 is not intended to be a single design tool. Other design charts available from the literature could also be used.[9, 10, 11, 12]

The design procedure in using the charts is as follows:

- Determine T_{S-MAX} using Figure 18 part A.
- Determine L_T and L_B using Figure 18 part B.
- Determine the distribution of reinforcement:

  -For low slopes ($H \leq 6$m) assume a uniform reinforcement distribution, and use T_{S-MAX} to determine spacing or the required tension T_{MAX} requirements for each reinforcement layer.

  -For high slopes ($H > 6$ m), divide the slope into two (top and bottom) or three (top, middle, and bottom) reinforcement zones of equal height, and use a factored T_{S-MAX} in each zone for spacing or design tension requirements.

  For 2 zones:

$$T_{Bottom} = 3/4\ T_{S-MAX} \qquad (11)$$

$$T_{Top} = 1/4\ T_{S-MAX} \qquad (12)$$

For 3 zones:

$$T_{Bottom} = 1/2\ T_{S-MAX} \qquad (13)$$

$$T_{Middle} = 1/3\ T_{S-MAX} \qquad (14)$$

$$T_{Top} = 1/6\ T_{S-MAX} \qquad (15)$$

The force is assumed to be uniformly distributed over the entire zone.

Determine reinforcement vertical spacing, S_v, or the maximum design tension, T_{MAX}, requirements for each reinforcement layer.

-For each zone, calculate T_{MAX} for each reinforcing layer in that zone based on an assumed S_v or, if the allowable reinforcement strength is known, calculate the minimum vertical spacing and number of reinforcing layers, N, required for each zone based on

$$T_d = T_a R_c = \frac{T_{zone} S_v}{H_{zone}} = \frac{T_{zone}}{N} \tag{16}$$

where:

R_c = coverage ratio of the reinforcement that equals the width of the reinforcement b divided by the horizontal spacing S_h.

S_v = vertical spacing of reinforcement in meters; multiples of compacted layer thickness for ease of construction.

T_{zone} = maximum reinforcement tension required for each zone.
= $T_{S\text{-}MAX}$ for low slopes (H< 6m).

H_{zone} = height of zone.
= T_{top}, T_{middle}, and T_{Bottom} for high slopes (H > 6m)

N = number of reinforcement layers.

Use short (1.2 – 2 m) lengths of intermediate reinforcement layers to maintain a maximum vertical spacing of 400 mm (16 in.) or less for face stability and compaction quality. Intermediate reinforcement should be placed in continuous layers and need not be as strong as the primary reinforcement, but it must be strong enough to survive construction (e.g., minimum survivability requirements for geotextiles in road stabilization applications in AASHTO M-288) and provide localized tensile reinforcement to the surficial soils.

For detailed analyses required for final design, refer to FHWA-NHI-00-043, *Mechanically Stabilized Earth Walls and Reinforced Soil Slopes, Design and Construction Guidelines.*

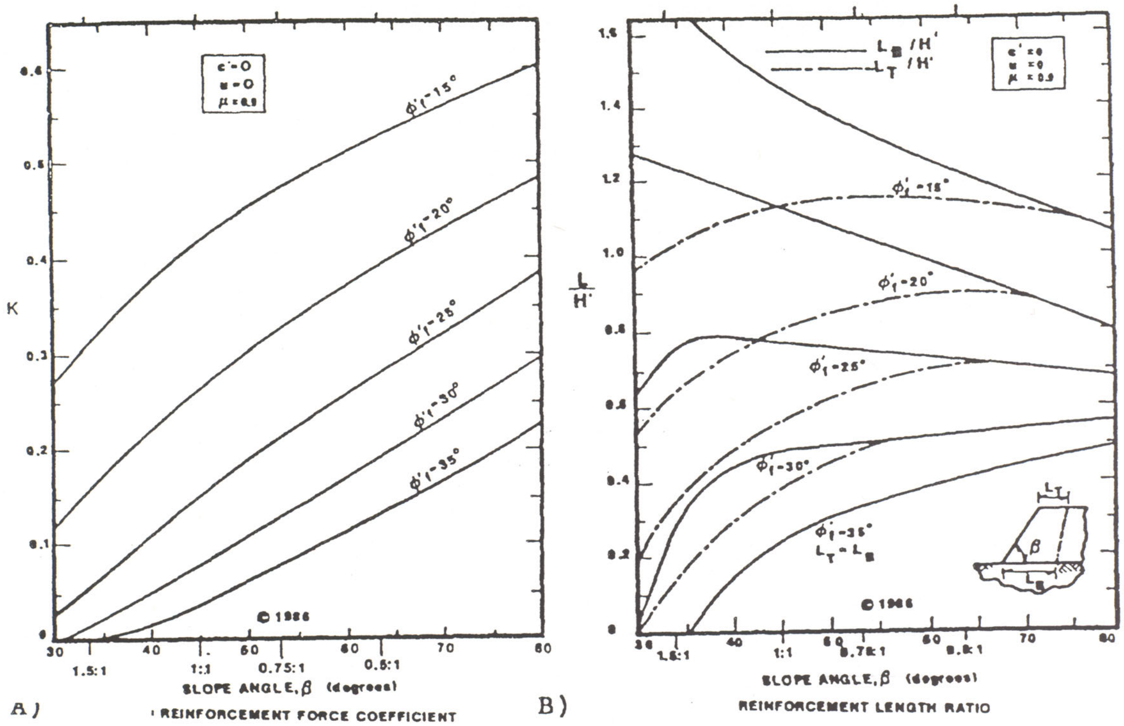


CHART PROCEDURE:

1. Determine force coefficient K from figure A above where:

$$\phi_f = \tan^{-1}\left(\frac{\tan \phi_r}{FS_R}\right)$$

where: ϕ_r = friction angle of reinforced fill

2. Determine:

$$T_{s\text{-}max} = 0.5\,K\,\gamma_r\,H'^2$$

where: $H' = H + q/\gamma_r$
q = a uniform surcharge

3. Determine the required reinforcement length at the top L_T and bottom L_B of the slope from figure B.

Limiting Assumptions:

- Extensible reinforcement.
- Slopes constructed with uniform, cohesionless soil (c=0).
- No pore pressures within slope.
- Competent, level foundation soils.
- No seismic forces.
- Uniform surcharge no greater than $0.2\gamma_r H$.
- Relatively high soil/reinforcement interface friction angle $\phi'_{sr} = 0.9\,\phi_r$ (may not be appropriate for some geotextiles).

Figure 18. Chart Solution for Determining the Reinforcement Strength Requirements (after Schmertmann, et. al., 1987).[9]

NOTE: Charts © The Tensar Corporation

d. Surficial Stability

For slopes flatter than 1H:1V, closer spaced reinforcements (i.e., every lift or every other lift, but no greater than 400 mm) preclude having to wrap the face in well graded soils (e.g., sandy gravel and silty and clayey sands). Wrapped faces are required for steeper slopes and uniformly graded soils to prevent face sloughing. Alternative vertical spacings could be used to prevent face sloughing, but in these cases, a face stability analysis should be performed either using the method presented in this chapter or by evaluating the face as an infinite slope using [10]

$$F.S. = \frac{c' H + (Y_g - Y_w) Hz \cos^2 \beta \tan \Phi' + F_g (\cos \beta \sin \beta + \sin^2 \beta \tan \Phi')}{Y_g Hz \cos \beta \sin \beta} \tag{17}$$

where:

c’	=	effective cohesion
ϕ'	=	effective friction angle
γ_g	=	saturated unit weight of soil
γ_w	=	unit weight of water
z	=	vertical depth to failure plane defined by the depth of saturation
H	=	vertical slope height
β	=	slope angle
F_g	=	summation of geosynthetic resisting force

e. Facing Options

Slope facing requirements will depend on soil type, slope angle, and reinforcement spacing. Table 9 may be used as a guide in selection. Discussion on grass type vegetation, soil bioengineering, and armored facings follows this table.

Grass-Type Vegetation

Stability of a slope can be threatened by erosion due to surface water runoff. Erosion control and revegetation measures must, therefore, be an integral part of all reinforced slope system designs and specifications. If not otherwise protected, reinforced slopes should be vegetated after construction to prevent or minimize erosion due to rainfall and runoff on the face. Vegetation requirements will vary by geographic and climatic conditions and are, therefore, project specific.

Table 9. RSS slope facing options

Slope Face Angle And Soil Type	Type of Facing			
	When Geosynthetic is not Wrapped at Face		When Geosynthetic is Wrapped at Face	
	Vegetated Face[1]	Hard Facing[2]	Vegetated Face[1]	Hard Facing[2]
> 50^o (> ~0.9H:1V) All Soil Types	Not Recommended	Gabions	Sod Permanent Erosion Blanket w/ Seed	Wire Baskets Stone Shotcrete
35^o to 50^o (~ 1.4H:1V to 0.9H:1V) Clean Sands (SP)[3] Rounded Gravel (GP)	Not Recommended	Gabions Soil-Cement	Sod Permanent Erosion Blanket w/ Seed	Wire Baskets Stone Shotcrete
35^o to 50^o (~ 1.4H:1V to 0.9H:1V) Silts (ML) Sandy Silts (ML)	Bioreinforcement Drainage Composites[4]	Gabions Soil-Cement Stone Veneer	Sod Permanent Erosion Blanket w/ Seed	Wire Baskets Stone Shotcrete
35^o to 50^o (~ 1.4H:1V to 0.9H:1V) Silty Sands (SM) Clayey Sands (SC) Well graded sands and gravels (SW & GW)	Temporary Erosion Blanket w/ Seed or Sod Permanent Erosion Mat w/ Seed or Sod	Hard Facing Not Needed	Geosynthetic Wrap Not Needed	Geosynthetic Wrap Not Needed
25^o to 35^o (~ 2H:1V to 1.4H:1V) All Soil Types	Temporary Erosion Blanket w/ Seed or Sod Permanent Erosion Mat w/ Seed or Sod	Hard Facing Not Needed	Geosynthetic Wrap Not Needed	Geosynthetic Wrap Not Needed

Notes: 1. *Vertical spacing of reinforcement (primary/secondary) shall be no greater than 400 mm, with primary reinforcements spaced no greater than 800 mm when secondary reinforcement is used.*
2. *Vertical spacing of primary reinforcement shall be no greater than 800 mm.*
3. *Unified Soil Classification.*
4. *Geosynthetic or natural horizontal drainage layers to intercept and drain the saturated soil at the face of the slope.*

For the unwrapped face (the soil surface exposed), erosion control measures are necessary to prevent unraveling and sloughing of the face. A wrapped face helps reduce erosion problems; however, treatments are still required on the face to shade the geosynthetic and prevent ultraviolet light exposure that will degrade the geosynthetic over time. In either case, conventional vegetated facing treatments generally rely on low-growth, grass-type vegetation, with more costly flexible armor occasionally used where vegetation cannot be established. Geosynthetic reinforced slopes can be difficult sites to establish and maintain grass-type vegetative cover due to the steep grades that can be achieved. The steepness of the grade limits the amount of water absorbed by the soil before runoff occurs. Although root penetration should not affect the reinforcement, the reinforcement will most likely restrict root growth. This can have an adverse influence on the growth of some plants. Grass is also frequently ineffective where slopes are impacted by waterways.

A synthetic (permanent) erosion control mat is normally used to improve the performance of grass cover. This mat must also be stabilized against ultra-violet light and should be inert to naturally occurring soil-born chemicals and bacteria. The erosion control mat serves to 1) protect the bare soil face against erosion until the vegetation is established; 2) assist in reducing runoff velocity for increased water absorption by the soil, thus promoting long-term survival of the vegetative cover; and 3) reinforce the surficial root system of the vegetative cover.

Once vegetation is established on the face, it must be protected to ensure long-term survival. Maintenance issues, such as mowing, must also be carefully considered. The shorter, weaker root structure of most grasses may not provide adequate reinforcement and erosion protection. Grass is highly susceptible to fire, which can also destroy the synthetic erosion control mat. Downdrag from snow loads or upland slides may also strip matting and vegetation off the slope face. The low erosion tolerance combined with other factors previously mentioned creates a need to evaluate revegetation measures as an integral part of the design. Slope face protection should not be left to the construction contractor or vendor's discretion. Guidance should be obtained from maintenance and regional landscaping groups in the selection of the most appropriate low-maintenance vegetation.

Soil Bioengineering (Woody Vegetation)

An alternative to low-growth, grass-type vegetation is the use of soil bioengineering methods to establish hardier, woody-type vegetation on the face of the slope [14]. Soil bioengineering uses living vegetation purposely arranged and imbedded in the ground to prevent shallow mass movement and surficial erosion. However, the use of soil bioengineering in itself is limited to stable slope masses. Combining this highly erosive system with geosynthetic reinforcement produces a very durable, low-maintenance structure with exceptional aesthetic and environmental qualities.

Appropriately applied, soil bioengineering offers a cost-effective and attractive approach for stabilizing slopes against erosion and shallow mass movement, capitalizing on the benefits and advantages that vegetation offers. The value of vegetation in civil engineering and the role woody vegetation plays in the stabilization of slopes has gained considerable recognition in recent years [15]. Woody vegetation improves the hydrology and mechanical stability of slopes through root reinforcement and surface protection. The use of deeply-installed and rooted woody plant materials, purposely arranged and imbedded during slope construction offers

- immediate erosion control for slopes; stream, and shoreline;
- improved face stability through mechanical reinforcement by roots;
- reduced maintenance costs, with less need to return to revegetate or cut grass;
- modification of soil moisture regimes through improved drainage and depletion of soil moisture and increase of soil suction by root uptake and transpiration;
- enhanced wildlife habitat and ecological diversity; and
- improved aesthetic quality and naturalization.

The biological and mechanical elements must be analyzed and designed to work together in an integrated and complementary manner to achieve the required project goals. In addition to using engineering principles to analyze and design the slope stabilization systems, plant science and horticulture are needed to select and establish the appropriate vegetation for root reinforcement, erosion control, aesthetics, and the environment. Numerous areas of expertise must integrate to provide the knowledge and awareness required for success. RSS systems require knowledge of the mechanisms involving mass and surficial stability of slopes. Likewise, when the vegetative aspects are appropriate to serve as reinforcements and drains, an understanding of the hydraulic and mechanical effects of slope vegetation is necessary.

Refer to FHWA-NHI-00-043, *Mechanically Stabilized Earth Walls and Reinforced Soil Slopes, Design and Construction Guidelines* for additional discussions on vegetation selection, vegetation placement, vegetation development, and design issues.

Armored

A permanent facing, such as gunite or emulsified asphalt may be applied to an RSS slope face to provide long-term ultra-violet protection, if the geosynthetic UV resistance is not adequate for the life of the structure. Welded wire mesh or gabions may also be used to facilitate face construction and provide permanent facing systems.

- Other armored facing elements may include riprap, stone veneer, articulating modular units, or fabric-formed concrete.
- Structural elements.

CHAPTER 5
CONTRACTING METHODS AND SPECIFICATIONS FOR MSE WALLS AND SLOPES

Since the early 1980s, hundreds of millions of dollars have been saved on our nation's highways by bidding **alternates** for selection of earth retaining structures. During that time, the number of available MSE systems or components, and the frequency of design and construction problems, have increased. Some problem areas that have been identified include misapplication of wall technology; poor specifications; lack of specification enforcement; inequitable bidding procedures; and inconsistent selection, review, and acceptance practices on the part of public agencies. Although the actual causes of each particular problem are unique, the lack of formal agency procedures that address the design and construction of earth retaining systems has repeatedly been an indirect cause.

MSE wall and RSS systems are contracted using two different approaches:

- Agency or material supplier designs with system components, drainage details, erosion measures, and construction execution explicitedly specified in the contracting documents; or

- Performance or end-result approach using approved or generic systems or components, with lines and grades noted on the drawings and geometric and design criteria specified. In this case, a project-specific design review and detail plan submittal occurs in conjunction with a normal working drawing submittal.

Some user agencies prefer one approach over the other or a mixed use of approaches, based upon the criticality of a particular structure. Both contracting approaches are valid if properly implemented. Each approach has advantages and disadvantages.

This chapter will outline the necessary elements of each contracting procedure, the approval process, and the current material and construction specifications.

While this chapter specifically addresses the need for formal policy and procedures for MSE and RSS structures, the recommendations and need for uniformity of practice applies to all types of retaining structures.

5.1 POLICY DEVELOPMENT

It is desirable that each agency develop a formal policy with respect to design and contracting of MSE wall and RSS systems. The general objectives of such a policy are to

- Obtain agency uniformity.
- Establish standard policies and procedures for design, technical review, and acceptance of MSEW and RSS systems or components.
- Establish responsibility for the acceptance of new retaining wall and reinforced slope systems and or components.
- Delineate responsibility in house for the preparation of plans, design review, and construction control.
- Delineate design responsibility for plans prepared by consultants and material suppliers.
- Develop design and performance criteria standards to be used on all projects.
- Develop and or update material and construction specifications to be used on all projects.
- Establish contracting procedures by weighing the advantages/disadvantages of proscriptive or end-result methods.

5.2 SYSTEM OR COMPONENT APPROVALS

The recent expiration of most process or material patents associated with MSE systems has led to the introduction of a variety of complete systems or components by numerous suppliers that are applicable for use. Alternatively, it opens the possibility of agency-generic designs that may incorporate proprietary and generic elements.

Approval of systems or components is a highly desirable feature of any policy for reinforced soil systems prior to their inclusion during the design phase, or as part of a value engineering alternate, subsequently offered.

For the purpose of prior approval, it is desirable that the supplier submit a data package that satisfactorily addresses the following items as a minimum:

- System development or component and year it was commercialized.
- Systems or component supplier organizational structure, specifically engineering and construction support staff.
- Limitations and disadvantages of system or component.
- Prior list of users, including contact persons, addresses and telephone numbers.
- Sample material and construction control specifications showing material type, quality, certifications, field testing, acceptance and rejection criteria, and placement procedures.
- A documented field construction manual describing in detail, with illustrations as necessary, the step-by-step construction sequence and the contractor's quality control plan.
- Detailed design calculations for typical applications in conformance with current practice or AASHTO, whenever applicable.
- Typical unit costs, supported by data from actual projects.
- Independent performance evaluations of a typical project by a professional engineer.

The development, submittal, and approval of such a technical package provides a complete benchmark for comparison with systems that have been in successful use, and offers a standard when checking project-specific designs.

5.3 DESIGN AND PERFORMANCE CRITERIA

It is highly desirable that each agency formalize its design and performance criteria as part of a design manual that may be incorporated in the *Bridge Design Manual* under *Retaining Structures for MSE walls* and/or a *Highway Design Manual* for reinforced slope structures. This would ensure that all designs, whether Agency/Consultant or Supplier prepared, are based on equal, sound principles.

The design manual may adopt current AASHTO Section 5.8 *Mechanically Stabilized Earth (MSE) Walls*, or methods outlined in FHWA-NHI-00-043, *Mechanically Stabilized Earth Walls and Reinforced Steepened Slopes, Design and Construction Guidelines* for both MSEW and RSS construction.

5.4 AGENCY OR SUPPLIER DESIGN

This contracting approach includes the development of a detailed set of MSE wall or RSS slope plans and material specifications in the bidding documents.

The advantage of this approach is that the complete design, details, and material specifications can be developed and reviewed over a much longer design period. This approach further empowers agency engineers to examine more options during design, but requires an engineering staff trained in MSE and RSS technology. This trained staff is also a valuable asset during construction when questions arise or design modifications are required.

The disadvantage is that for alternate bids, additional sets of designs and plans must be processed, although only one will be constructed. A further disadvantage is that newer and potentially less expensive systems or components may not be considered during the design stage.

The fully detailed plans shall include, but not be limited to, the following items:

a. Plan and Elevation Sheets

- Plan view to reflect the horizontal alignment and offset from the horizontal control line to the face of wall or slope. Beginning and end stations for the reinforced soil construction and transition areas, and all utilities, signs, lights, etc., that affect the construction should be shown.

- For MBW unit faced walls, the plan view should show alignment baseline, limits of bottom of wall alignment, and limits of top of wall alignment, as alignments vary with the batter of MBW system actually supplied.

- Elevation views indicating elevations at top and bottom of walls or slopes, beginning and end stations, horizontal and vertical break points, location and elevation of copings and barriers, and whole station points. Location and elevation of final ground line shall be indicated.

- Length, size, and type of soil reinforcement and where changes in length or type occur shall be shown.

- Panel and MBW unit layout and the designation of the type or module, the elevation of the top of leveling pad and footings, the distance along the face of the wall to all steps in the footings and leveling pads.

- Internal drainage alignment, elevation, and method of passing reinforcements around such structures.

- Any general notes required for construction.

- Cross sections showing limits of construction, fill requirements, and excavation limits. Mean high water level, design high water level, and drawdown conditions shall be shown, where applicable.

- Limits and extent of reinforced soil volume.

- All construction constraints, such as staged construction, vertical clearance, right-of-way limits, etc.
- Payment limits and quantities.

b. Facing/Panel Details

- Facing details for erosion control for reinforced slopes and all details for facing modules, showing all dimensions necessary to construct the element, reinforcing steel, and the location of reinforcing attachment devices embedded in the panels.

- All details of the architectural treatment or surface finishes.

c. Drainage Facilities/Special Details

- All details for construction around drainage facilities, overhead sign footings, and abutments.

- All details for connection to traffic barriers, copings, parapets, noise walls, and attached lighting.

- All details for temporary support including slope face support, where warranted.

d. Design Computations

- The plans shall be supported by detailed computations for internal and external stability and life expectancy for the reinforcement.

- For plans prepared by material suppliers, deep-seated global stability is normally determined by the Owner and/or their consultant. Responsibility for compound stability analysis, when applicable, must be defined by the Owner.

e. Geotechnical Report

The plans shall be prepared based on a geotechnical report that details the following:

- Engineering properties of the foundation soils, including shear strength and consolidation parameters used to establish settlement and stability potential for the proposed construction. Maximum bearing pressures must be established for MSE wall construction.

- Engineering properties of the reinforced soil, including shear strength parameters (ϕ, c) compaction criteria, gradation, and electrochemical limits.

- Engineering properties of the fill or in-situ soil behind the reinforced soil mass, including shear strength parameters (ϕ, c) and for fills compaction criteria.

- Groundwater or free water conditions and required drainage schemes.

f. Construction Specifications

Construction and material specifications for the applicable system or component as detailed in FHWA-NHI-00-043, *Mechanically Stabilized Earth Walls and Reinforced Steepened Slopes, Design and Construction Guidelines* for both MSEW and RSS construction.

5.5 END RESULT DESIGN APPROACH

Under this approach, often referred to as "line and grade" or "two line drawing," the agency prepares drawings of the geometric requirements for the structure or reinforced slope and material specifications for the components or systems that may be used. The components or systems that are permitted are specified or are from a pre-approved list maintained by the agency from its prequalification process.

The end-result approach, with sound specifications and prequalification of suppliers and materials, offers several benefits. Design of the MSE structure is performed by trained and experienced staff. The prequalified material components (facing, reinforcement, and miscellaneous) have been successfully and routinely used together, which may not be the case for in-house design with generic specifications for components. Also, the system specification approach lessens engineering costs and manpower for an agency, and transfers some of the project's design cost to construction.

The disadvantage is that agency engineers may not fully understand the technology at first and, therefore, may not be fully qualified to review and approve construction modifications. Newer and potentially less expensive systems may not be considered due to the lack of confidence of agency personnel to review and accept these systems. In addition, complex phasing and special details are not addressed until after the contract has been awarded.

The bid quantities are obtained from specified pay limits denoted on the "line and grade" drawings and can be bid on a lump-sum or unit-price basis. The basis for detailed designs to be submitted after contract award are contained either as complete special provisions or by reference to AASHTO or agency manuals, as a special provision.

Plans, furnished as part of the contract documents, contain the geometric, geotechnical, and design-specific information listed below:

a. Geometric Requirements

- Plan and elevation of the areas to be retained, including beginning and end stations.

- For MBW unit faced walls, the plan view should show alignment baseline, limits of bottom of wall alignment, and limits of top of wall alignment, as alignments vary with the batter of MBW system actually supplied.

- Typical cross-section that indicates face batter, pay limits, drainage requirements, excavation limits, etc.

- Elevation view of each structure, showing original ground line, minimum foundation level, finished grade at ground surface, and top of wall or slope line.

- Location of utilities, signs, etc., and the loads imposed by each such appurtenance, if any.

- Construction constraints, such as staged construction, right-of-way, construction easements, etc.

- Mean high water level, design high water level, and drawdown conditions, where applicable.

b. Geotechnical Requirements

They are the same as in Section 5.4, except that the design responsibility is clearly delineated as to areas of contractor/supplier and agency responsibility.

Typically, the agency would assume design responsibility for developing stability, allowable bearing and settlement analyses, as they would be the same regardless of the system used. The contractor/supplier would assume responsibility for both internal and external stability for the designed structures.

c. Structural and Design Requirements

- Reference to specific governing sections of the agency design manual (materials, structural, hydraulic, and geotechnical), construction specifications, and special provisions. If none is available for MSE walls, refer to current AASHTO, both Division I, Design, and Division II, Specifications.

- Magnitude, location, and direction of external loads due to bridges, overhead signs and lights, and traffic surcharges.

- Limits and requirements of drainage features beneath, behind, above, or through the reinforced soil structure.

- Slope erosion protection requirements for reinforced slopes.

- Size and architectural treatment of concrete panels for MSE walls.

d. Performance Requirements

- Tolerable movement of the structure, both horizontal and vertical.

- Tolerable face panel movement.

- Monitoring and measurement requirements.

5.6 REVIEW AND APPROVALS

Where agency design is based on supplier's plans, it should be approved for incorporation into the contract documents following a rigorous evaluation by agency structural and geotechnical engineers. The following is a checklist of items requiring review:

- Conformance to the project line and grade.

- Conformance of the design calculations to agency standards or codes, such as current AASHTO with respect to design methods, allowable bearing capacity, allowable tensile strength, connection design, pullout parameters, surcharge loads, and factors of safety.

- Development of design details at obstructions, such as drainage structures or other appurtenances, traffic barriers, cast-in-place junctions, etc.

- Facing details and architectural treatment.

For end-result contracting methods, the special provisions should contain a requirement that complete design drawings and calculations be submitted within 60 days of contract award for agency review.

The review process should be similar to the supplier design outlined above and be conducted by the agency's structural and geotechnical engineers.

5.7 CONSTRUCTION SPECIFICATIONS AND SPECIAL PROVISIONS FOR MSE AND RSS CONSTRUCTION

A successful reinforced soil project will require sound, well-prepared material and construction specifications to communicate project requirements, as well as construction guidance to both the contractor and inspection personnel. Poorly prepared specifications often result in disputes between the contractor and owner representatives.

A frequently occurring problem with MSE systems is the application of different or unequal construction specifications for similar MSE systems. Users are encouraged to utilize a single unified specification that applies to all systems, regardless of the contracting method used. The construction and material requirements for MSE systems are sufficiently well developed and understood to allow for unified material specifications and common construction methods.

For guide construction and material specifications, refer to FHWA-NHI-00-0043, *Mechanically Stabilized Earth Walls and Reinforced Steepened Slopes, Design and Construction Guidelines.*

5.8 CONSTRUCTION CONTROL

Prior to construction of any MSE structure, personnel responsible for construction control should become familiar with the following items:

- plans, specifications and testing requirements
- site conditions relevant to construction conditions
- material requirements and allowable tolerances
- construction sequencing

Quality assurance measures must be implemented to:

- Inspect reinforcement for damage and availability of mill test certificates to certify grade and corrosion protection, where required for metallic reinforcement and compliance with submitted manufacturer's specifications for geosynthetics.

- Inspect precast elements for damage and casting tolerances.

- Inspect and test reinforced backfill for conformance to gradation and electrochemical properties.

During erection, construction control is focused on assuring that:

- Leveling pads are correctly constructed with respect to elevation.

- Panels are erected within the specified vertical tolerances.

- The fill is compacted to the required uniform density within the required moisture content range.

- The reinforcements are properly secured to the facing panels.

- The vegetative cover for RSS structure is correctly installed.

CHAPTER 6
COST DATA

Site-specific costs of a soil-reinforced structure are a function of many factors, including cut-fill requirements, wall/slope size and type, in-situ soil type, available backfill materials, facing finish, temporary or permanent application. It has been found that MSE walls with precast concrete facings are usually less expensive than reinforced concrete retaining walls for heights **greater** than about 3 m (10 ft.) and average foundation conditions. Modular block walls (MBW) are competitive with concrete walls at heights of **less** than 4.5 m (15 ft.).

In general, the use of MSE walls results in savings on the order of 25 – 50% and possibly more in comparison with a conventional reinforced concrete retaining structure, especially when the latter is supported on a deep foundation system (poor foundation condition). A substantial savings is obtained by elimination of the deep foundations, which is usually possible because reinforced soil structures can accommodate relatively large total and differential settlements. Other cost saving features include ease of construction and speed of construction. A comparison of wall material and erection costs for several reinforced soil retaining walls and other retaining wall systems, based on a survey of state and federal transportation agencies, is shown in Figure 19. Typical total costs for MSE walls range from \$200 – \$400 per m^2 (\$19 – \$37 per $ft.^2$) of face, generally as function of height, size of project, and cost of select fill.

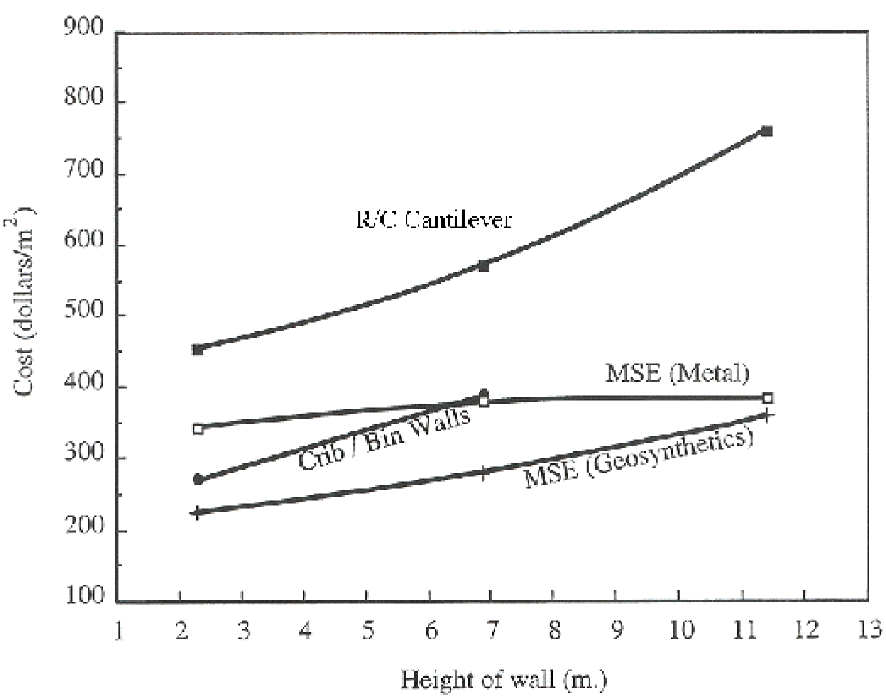


Figure 19. Cost Comparison for Retaining Walls.[12]

The actual cost of a specific MSEW structure will depend on the cost of each of its principal components. For segmental precast concrete faced structures, typical relative costs are

- erection of panels and contractors profit: 20 – 30% of total cost
- reinforcing materials: 20 – 30% of total cost
- facing system: 25 – 30% of total cost
- backfill materials, including placement: 35 – 40% of total cost, where the fill is a select granular fill from an off-site borrow source

The additional cost for panel architectural finish treatment ranges from \$5 – \$15 per m^2 (\$0.50 – \$1.50 per $ft.^2$), depending on the complexity of the finish. Traffic barrier costs average \$550 per linear meter (\$170 per linear foot). In addition, consideration must be given to the cost of excavation, which may be somewhat greater than for other systems. MBW faced walls at heights less than 4.5 m (15 ft.) are typically less expensive by 10% or more.

The economy of using RSS must be assessed on a case-by-case basis, where use is not dictated by space constraints. For such cases, an appropriate benefit to cost ratio analysis should be carried out to see if the steeper slope with the reinforcement is justified economically over the alternative flatter slope with its increased right-of-way and materials costs, etc. It should be kept in mind that guardrails or traffic barriers are often necessary for steeper embankment slopes, and additional costs, such as erosion control systems for slope face protection, must be considered.

With respect to economy, the factors to consider are as follows

- cut or fill earthwork quantities
- size of slope area
- average height of slope area
- angle of slope
- cost of nonselect versus select backfills

- temporary and permanent erosion protection requirements
- cost and availability of right-of-way needed
- complicated horizontal and vertical alignment changes
- need for temporary excavation support systems
- maintenance of traffic during construction
- aesthetics
- requirements for guardrails and traffic barriers

The actual bid cost of a specific RSS structure depends on the cost of each of its principal components. Based on limited data, typical relative costs are

- Reinforcement 45 – 65% of total cost
- Backfill 30 – 45% of total cost
- Face treatment 5 – 10% of total cost

High RSS structures have relatively higher reinforcement and lower backfill costs. Recent bid prices suggest costs ranging from $110/m^2 – $260/m^2 ($10/ft.^{2} – $24/ft.^{2}) as a function of height.

For applications in the 10 – 15 m (30 – 50 ft.) height range, bid costs of about $170/m^2 ($16/ft.^{2}) have been reported. These prices do <u>not</u> include safety features and drainage details.

CHAPTER 7
CASE HISTORIES

7.1 MSE CONSTRUCTION

The following case histories are presented to provide representative examples of cost-effective, successful MSE applications. All project information was obtained from the indicated reference, which, in most cases, contains additional details.

a. I-94 Wall using MBW Facing Units[3]

The first segmental block-faced MSE wall specified by MnDOT was constructed on the south side of Interstate 94, immediately west of Western Avenue in the city of St. Paul. The wall is nearly 180 m (600 ft.) long and has a maximum height of 4.3 m (14 ft.). The completed structure is shown in Figure 20.

During conceptual design, a low, gravity wall 1 – 1.5 m (3.3 – 4.9 ft.) was envisioned at this location, with retention provided by MBW units, without soil reinforcement elements. During final design, heights up to 3.2 m (10.5 ft.) became necessary, and the contract documents, therefore, included a contractor's design for an MSE wall, with MBW facing units to accommodate this retention height plus an embedment below the anticipated frost line at this location. The resulting final maximum wall height was on the order of 4.3 m (14 ft.).

Design

The project specifications required that the contractor provide a design in accordance with the manufacturer's guidelines, as no agency specifications for design existed at that time, and that the fill within the reinforced zone be on-site soils that would meet the MnDOT requirements for granular borrow. These requirements specify a maximum size of 25 mm (1 in.) and no more than 20% passing the #200 sieve.

The contractor submitted a design in accordance with guidelines published by Task Force 27 of AASHTO-AGC-ARTBA. The design submitted used a proprietary MBW unit and a high density polyethylene geogrid reinforcement with a length typically at 80% of the height. The minimum vertical spacing was 200 mm (8 in.) at the bottom of the wall, with 600 mm (24 in.) spacing at the top.

Figure 20. Completed I-94 Wall.

This maximum spacing was based on temporary stability of the facing blocks during construction. The geogrid reinforcement strength was determined by applying reduction factors to the ultimate strength of the reinforcement. These factors for creep, aging, and construction damage were developed by the material supplier from data in his files. The connection strength between the reinforcement and the blocks was based on test data specifically developed for these blocks at the University of Wisconsin - Platteville.

Erection

Base blocks were laid in 50-foot chord lengths. A granular soil leveling pad was placed and block laid directly on top. The base block was laid in an inverted position so the lip was on top and at the front. Horizontal alignment was controlled with reference to the back of the base block lip. Subsequent blocks were laid in the normal position.

The blocks have an automatic 165 mm per meter (2 in. per ft.) batter from the trailing lip. The top of the wall at the tallest section of 4.3 m (14 ft.) was, therefore, 710 mm (28 in.) off plan wall alignment. This batter increased stability of the wall, but was not accounted for in the analysis procedure. The wall also stepped back from the planned horizontal wall alignment 165 mm (6.5 in.), with each meter raise in the footing elevation. This setback did not create any problems, but specifying agencies and designers should be aware that setbacks vary with segmental block types, and they should consider such when specifying and designing.

Level of the segmental blocks along the wall alignment was controlled with a carpenter level, as the blocks were laid and checked intermittently with survey points. Some problems with holding the blocks level in the direction perpendicular to wall alignment were experienced. An approximate 3-m-wide (10 ft.) section of wall that bowed outward when the wall was about 3 m (10 ft.) high was observed on the first section of wall built. The bow was eliminated by removing the facing block, drainage layer, and geotextile, and reerecting them with adjustments to the alignment. The reinforced soil mass, geogrids, and soil fill stood vertically for 2 days, as this reconstruction was completed.

Conclusions

Based on the experience with this first project, Mn DOT concluded that on future projects:

- Contractor's design is to be governed by all applicable requirements of AASHTO, Section 5.8 as amended by MnDOT.

- Wall systems to be used on future projects must have pre-approved components by MnDOT, and the contractor must identify his chosen system in the bid.

- Standards for the manufacture of MBW blocks with respect to compressive strength and water absorption must be developed by MnDOT.

- Erection tolerances on the order of 25 mm per 3 m (1 in. per 10 ft.) of height must be included in the specifications.

Cost

The bid cost was \$270 per square meter (\$25/ft.^{2}) installed. Cast-in-place concrete walls on the same project were bid at \$410 per square meter (\$38/ft.^{2}).

b. US 23 (I-18 Extension) Unicoi Co. TN, using Precast Concrete Facing Units

The use of superimposed MSE structures with precast concrete facings for a combined height in excess of 30 m (100 ft.) was specified by Tennessee DOT on a construction contract on US 23 in eastern Tennessee. Construction was successfully completed in the fall of 1994. The project from Erwin, TN, to the North Carolina border is presently designated as a widening of US 23, but eventually will be incorporated as an extension of I-26 from Ashville, NC, to Johnson City, TN.

The relocation and widening of this route through the rugged terrain of the Great Smoky Mountains required massive rock cuts and high fills over steep and narrow ravines. The construction contract specified the construction of retaining walls at multiple locations, including locations requiring up to three levels of superimposed walls. The total retained height, including fill slopes above the uppermost wall, was in excess of 60 m (200 ft.).

Tennessee DOT developed the geometry of retention at each location, specifying the number and location of tiers, setbacks, height of each wall, length and height of fill slopes above the wall, and material and construction specifications. The select backfill source was specified as processed on-site gneiss rock, from cut areas, crushed, and screened to a maximum size of 75 mm (3 in.).

The construction contract contained alternate MSE system designs prepared by proprietary MSE suppliers, based on the required geometry and design parameters developed by Tennessee DOT. The designs prepared in advance to bidding, and included in the bid documents, were based on design criteria developed by the agency, which included strength and frictional parameters for the various fills utilized, bearing capacity, and factors of safety. The design method was not specified, due to the lack of uniform standards at the time, and each supplier followed its design practice.

The submitted designs were checked for compliance to the geometric and design requirements and the unique pay quantities included in the bid summary sheets. The successful bidder was required to specify the system chosen to preclude post-bid shopping by the general contractor. The low bid price was \$313/m^2 (\$29/ft.^{2}), which included the placement and compaction of the select fill within the reinforced soil zone. The completed construction at a tiered location shown in Figure 21, proceeded with no difficulty, at an average erection rate of 60 square meters (645 ft.^{2}) per day.

a) During erection

b) Completed

Figure 21. US 23 Superimposed MSE Walls.

7.2 RSS CONSTRUCTION

The following case histories are presented to provide representative examples of cost-effective, successful reinforced slope projects. In several cases, instrumentation was used to confirm the performance of the structure.

a. The Dickey Lake Roadway Grade Improvement Project [4]

Dickey Lake is located in northern Montana, approximately 40 km south of the Canadian border. Reconstruction of a portion of U.S. 93 around the shore of Dickey Lake required the use of an earth-retention system to maintain grade and alignment. The fill soils available in the area consist primarily of glacial till. Groundwater is active in the area. A slope stability factor of safety criteria of 1.5 was established for the embankments. A global stability analysis of reinforced concrete retaining walls to support the proposed embankment indicated a safety factor that was less than required. Analysis of a reinforced soil wall or slope indicated higher factors of safety. Based on an evaluation of several reinforcement systems, a decision was made to use a reinforced slope for construction of the embankment. MDOT decided that the embankment would not be designed "in-house," due to their limited experience with this type of structure. Proposals were solicited from a variety of suppliers who were required to design the embankment. An outside consultant experienced in geosynthetic reinforcement design was retained to review all submittals.

Plans and specifications for the geosynthetic reinforced embankments(s) were developed by MDOT, with the plans indicating the desired finished geometry. The slopes generally ranged from 9 m – 18 m (30 – 60 ft.) in height. Face angles varied from 1.5H:1V to 0.84H:1V, with the typical angle being 1H:1V. The chosen supplier provided a design that utilized both uniaxially and biaxially oriented geogrids. The resulting design called for primary reinforcing grids 4.6 – 18.3 m (15 – 60 ft.) long, and spaced 0.6 – 1.2 m (2 – 4 ft.) vertically throughout the reinforced embankment. The ultimate strength of the primary reinforcement was on the order of 100 kN/m (6,850 plf). The length of primary reinforcement was partially dictated by global stability concerns. In addition, intermediate reinforcement consisting of lower strength, biaxial geogrids, was provided in lengths of 1.5 m (5 ft.), with a vertical spacing of 0.3 m (1 ft.) at the face of slopes 1H:1V or flatter. Erosion protection on the 1H:1V or flatter sections was accomplished by using an organic erosion blanket. Steeper sections (maximum 0.84H:1V) used L-shaped, welded-wire forms, with a biaxial grid wrap behind the wire.

The design also incorporated subsurface drainage. This drainage was judged to be particularly important, due to springs or seeps present along the backslope of the embankment. The design incorporated geocomposite prefabricated drains placed along the backslope, draining into a French drain at the toe of the backslope. Laterals extending under the embankment were used to "daylight" the French drain.

The project was constructed in 1989 at a cost of approximately \$180/m^2 of vertical face, and has been periodically monitored by visual inspection and slope inclinometers. Project photos are shown in Figure 22. To date, the embankment performance has been satisfactory, with no major problems observed. Some minor problems have been reported with respect to the erosion control measures, and some minor differential movement in one of the lower sections of the embankment has been noted.

b. Salmon-Lost Trail Roadway Widening Project [(5)]

As part of a highway widening project in Idaho, the Federal Highway Administration designed and supervised the construction of a 172-m-long (565 ft.), 15.3-m-high (50 ft.), permanent geosynthetic-reinforced slope to compare its performance with retaining structures along the same alignment. Widening of the original road was achieved by turning the original 2H:1V unreinforced slope into a 1H:1V reinforced slope. Aesthetic appeal was an important consideration in the selection of the retaining structures along scenic Highway 93, which has been recognized by a recent article in *National Geographic*. A vegetated facing was, therefore, used for the reinforced slope section. On-site soil consisting of decomposed granite was used as the backfill. An important factor in the design was to deal with seeps or weeps coming out of the existing slope. Geotextile reinforcements with an in-plane transmissivity were selected to evaluate the potential of modifying the seepage regime in the slope.

The geotextile-reinforced slope was designed in accordance with the guidelines presented in this technical summary. The final design consisted of two reinforced zones with a constant reinforcing spacing of 0.3 m (1 ft.). The reinforcement in the lower zone had an ultimate tensile strength of 100 kN/m (6,850 lb/ft.), and the reinforcement in the upper zone had a reinforcement strength of 20 kN/m (1,370 lb/ft.). The reinforcement strength was reduced based on partial reduction factors. Field tests were used to reduce the reduction factor for construction damage from 2.0 to 1.1, at a substantial savings to the project (40% reduction in reinforcement).

The construction was completed in 1993 (see Figure 23 for project photos). The structure was constructed as an experimental features project and was instrumented with inclinometers within the reinforced zone, extensometers on the reinforcement, and piezometers within and at the back of the reinforced section. Survey monitoring was also performed during construction. Total lateral displacements recorded during construction were on the order of 0.1 – 0.2% of the height of the slope, with maximum strains in the reinforcement measured at only 0.2%. Post-construction movement has not been observed within the accuracy of the instruments. These measurements indicate the excellent performance of the structure, as well as the conservative nature of the design. Long-term monitoring is continuing.

The steepened slope was constructed at a faster rate and proved more economical than the other retaining structures constructed along the same alignment. The constructed cost of the reinforced slope section was on the order of \$160/m^2 (\$15/ft.^{2}) of vertical face. MSEW costs in other areas of the site were on the order of \$240/m^2 (\$22/ft.^{2}) of vertical face, for similar or lower heights.

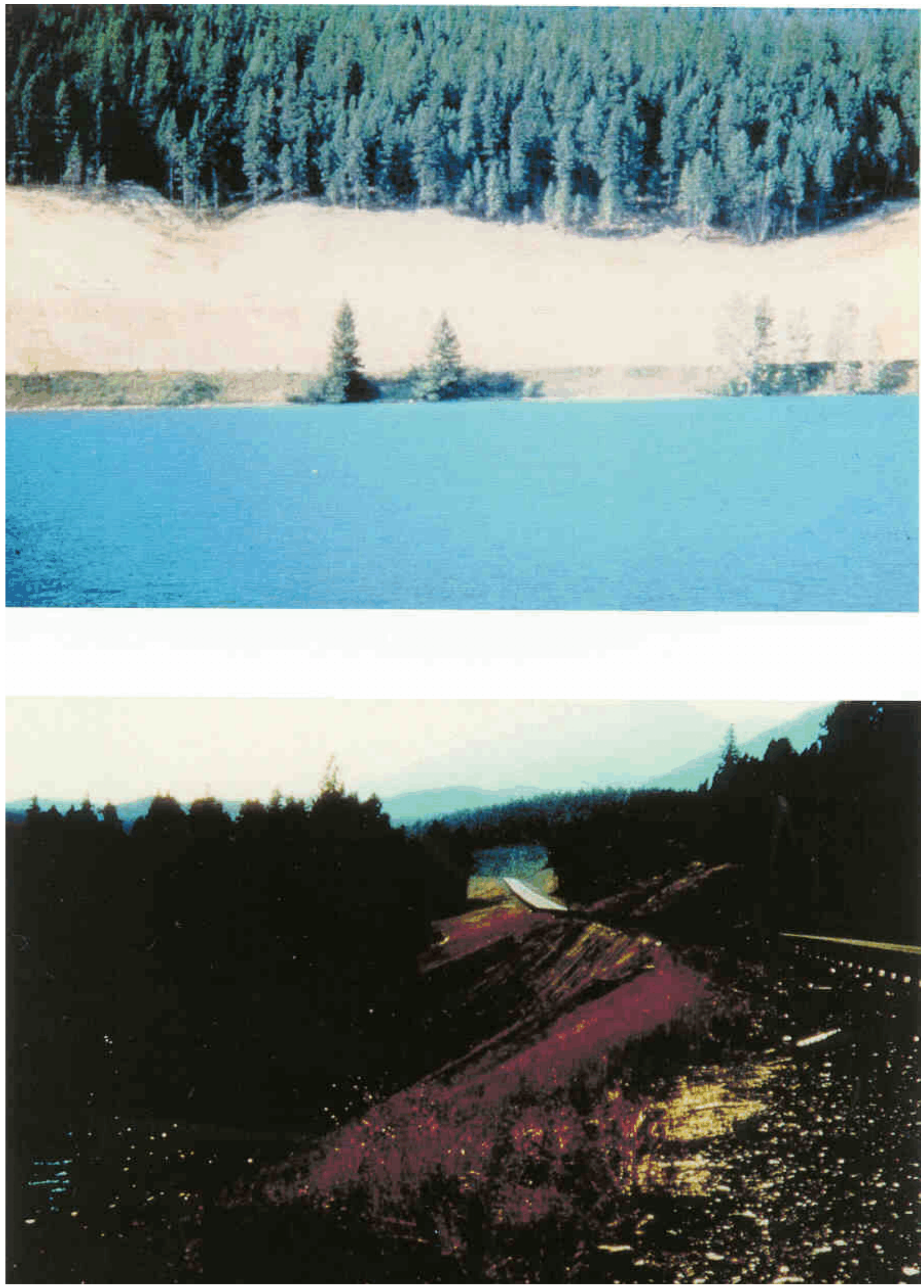


Figure 22. Dickey Lake Site.

Figure 23. Salmon-Lost Trail Site.

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Technical Summary # 9:

SOIL NAILING

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CHAPTER 1
DESCRIPTION AND HISTORY

1.1 PREFACE

This technical summary for soil nailing has been developed based on the following FHWA reports:

- *Soil Nailing for Stabilization of Highway Slopes and Excavations,* FHWA RD 89-198, 1989.

- *Recommendations Clouterre,* FHWA SA-93-026, 1991 (English Translation).

- *Soil Nailing Field Inspectors Manual,* FHWA SA-93-068, 1994.

- *Manual for Design and Construction Monitoring of Soil Nail Walls,* FHWA-SA-96-069, (Revised 1998).

- *Engineering Circular # 7, Soil Nail Walls,* FHWA IF-03-017. (1)

1.2 DESCRIPTION

Soil Nailing is a construction technique for reinforcing existing ground and constructing walls in cut sections. This is accomplished by installing closely spaced, passive, structural inclusions, known as nails, into the soils to increase their overall shear strength. The term "passive" means that the nails are not pre-tensioned when they are installed, as with tiebacks. The nails develop tension as the ground deforms laterally in response to continued excavation. Nails may be used to stabilize either existing slopes or future slopes/cuts created by excavation activities at a site. A structural facing connected to the nails is used to complete the work and to give it a finished appearance consistent with the project's aesthetic requirements.

This technique has been used for both temporary and permanent construction. The cost savings are obtained primarily from the expediency of construction and the structural benefits of distributing the developed earth pressure loads over a large number of nails.

A unique feature of soil nailed walls is that they are built from the top down, in small (typically two meters or less) successive lifts, as illustrated in Figure 1. The construction of each lift involves three basic steps that are repeated until the final depth is achieved. These steps are

1) Excavation
2) Nail installation
3) Shotcreting

Depending on ground conditions, steps 2 and 3 may be reversed. Permanent walls typically include an additional step consisting of placement of a permanent wall facing (cast-in-place concrete, precast elements, or additional shotcrete) over the initial shotcrete layer.

1.3 HISTORY

Soil nailing has been used in a variety of civil engineering projects in the last two decades. It appears that the technique has emerged as an extension of rock bolting and of the "New Austrian Tunneling Method" (NATM), which combines reinforced shotcrete and rock bolting to provide a flexible support system for the construction of underground excavations.

In North America, the system was initially used in Vancouver, B.C., Canada, in the early 1970s for temporary excavation support. In Europe, the earliest reported works were retaining wall construction in Spain (1972), France (1973), and Germany (1976), in connection with highway or railroad cut slope construction or temporary building excavation support. The long-term performance of soil nail walls has been proven after 30 years of use in Europe and the United States.

There has been significant utilization of this construction method in the United States in the last decade for both temporary and permanent construction, as well as research and development to refine design methods, construction techniques, and material specifications.

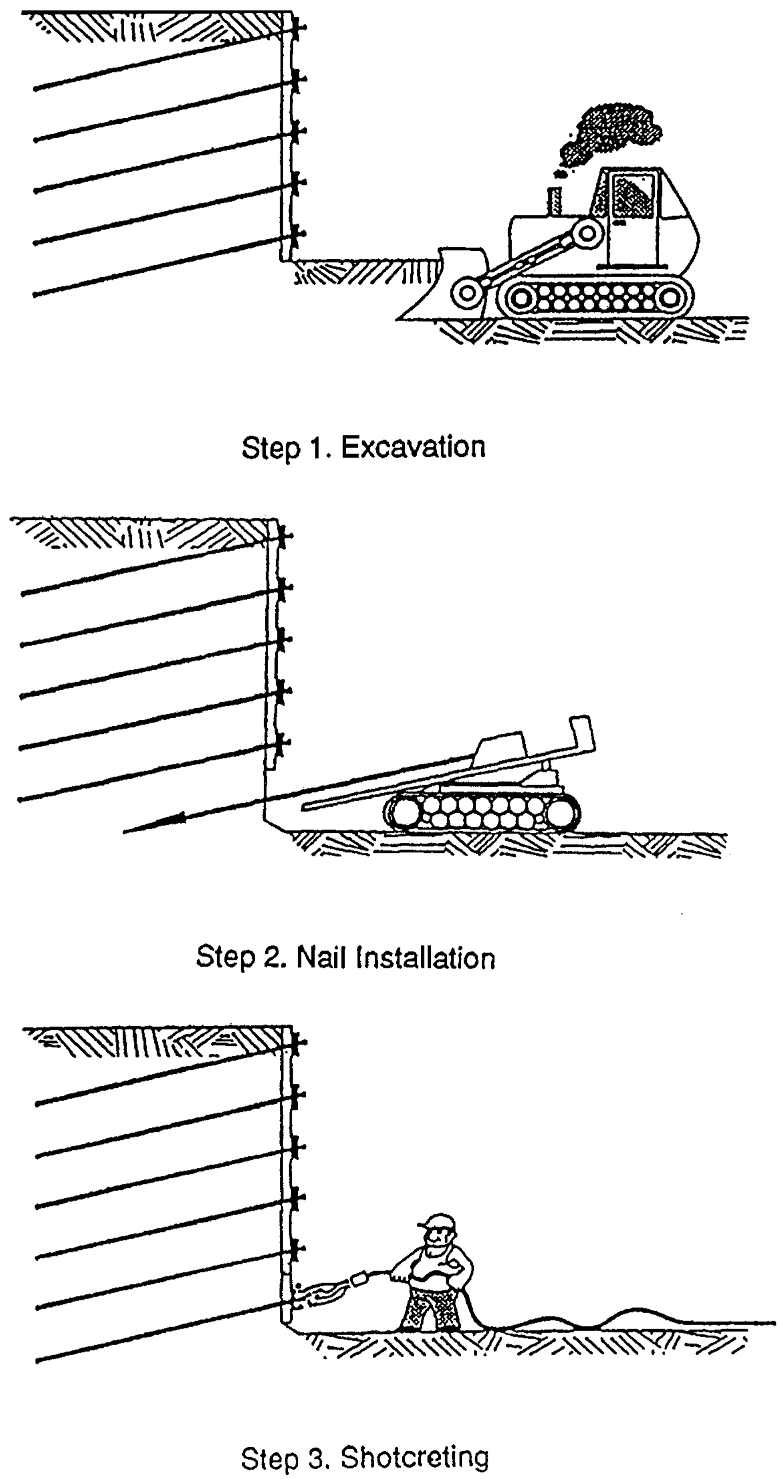


Figure 1. Soil Nail Wall Construction Sequence.

1.4 FOCUS AND SCOPE

The purpose of this document is to acquaint the reader with soil nailing by providing an overview of the technology. The items discussed include

- areas of application
- advantages/disadvantages
- feasibility evaluations
- preliminary design and construction sequencing
- bidding methods, construction specifications, and control
- case histories and cost data

This document does not provide in-depth recommendations on design or inspection control. In-depth design methods and construction specifications have been developed and are contained in FHWA-IF-017 "Geotechnical Engineering Circular # 7, Soil Nail Walls."[(1)]

For in-depth recommendations on field inspection and construction methods, the reader is additionally referred to FHWA SA-93-068 "Soil Nailing Field Inspection Manual".

CHAPTER 2
DESIGN CONSIDERATIONS

2.1 APPLICATIONS

Soil nailing technology can be advantageously considered for permanent or temporary cut wall application where conventional cast-in-place, precast, or mechanically stabilized earth (MSE) structures are applicable, but especially where tieback walls are considered or thought to be applicable. Soil nailing under certain circumstances is more cost effective than tieback construction and provides more redundancy in design. Savings compared to more conventional retaining techniques have been reported in the range of 10 – 30%. A wide range of aesthetic requirements can be accommodated by casting concrete in-place over the shotcrete facing, by texturing the shotcrete, or by using precast concrete elements suitably connected to the nails/shotcrete facing.

The following specific uses have been demonstrated in a variety of transportation-related projects constructed in North America and Europe:

- Vertical or near-vertical cut construction in both soils and weathered rock. Soil nailing minimizes excavation quantities, and reduces right-of-way needs and clearing limits, minimizing environmental impact within a transportation corridor.

  This usage is particularly applicable for widening projects that must be constructed within existing right-of-way.

- Tunnel portals located in steep terrain of variable stratigraphy (soil/rock) and, therefore, are subject to slope movements. Soil nailing has been used successfully to stabilize portals on a number of projects.

- Bridge end-slope removal. As with tiebacks, soil nailing is applicable in end-slope removal applications for bridges. The nails are installed during the excavation phase. For the more common case of underpass widening, nailing offers a major advantage in that it requires no soldier pile installation, which would be difficult to install under restricted headroom conditions, and can provide both the temporary and permanent earth support functions.

Soil nailing is also applicable to existing retaining structures requiring stabilization and strengthening. The types of projects under this category include repairs to

- masonry or reinforced concrete retaining walls after or just before failure caused by structural deterioration or excessive deflections.

- for anchored walls experiencing failure due to overloading or corrosion of anchor tendons.

- mechanically stabilized earth (MSE) walls to provide horizontal and mass stability lost due to corrosion of the strips or grids or poor quality backfill.

2.2 ADVANTAGES AND DISADVANTAGES

Advantages

Soil nailing has many advantages as compared to conventional reinforced concrete retaining walls, in addition to certain unique benefits. When used in connection with temporary or permanent cut construction or stabilization along the highway right-of-way or in connection with tunnel portal stabilization, these advantages can be summarized as follows:

- A temporary excavation system can be incorporated into the permanent facility.

- Cut excavation and rock blasting is reduced.

- Concrete quantities are reduced.

- Deep foundations for support are eliminated.

- Right-of-way acquisition and environmental impacts are potentially reduced.

- Cost is lower due to relatively rapid installation of the unstressed inclusions (nails) which are considerably shorter than ground anchors, and a relatively thin shotcrete or concrete facing.

- Only light construction equipment is required to install nails, as well as simple grouting equipment. Grouting of the boreholes is generally accomplished by gravity. This feature may be of particular importance for sites with difficult access.

- Since there is a high density of nails, failure of any one nail may not significantly affect the stability of the system.

- In heterogeneous soils with cobbles, boulders, and weathered or hard rock zones, it offers the advantage of small-diameter, shorter drill holes for nail installation, and eliminates the need for soldier pile installation that is disproportionately costly to install under these conditions.

- Soil nailed structures are more flexible than conventional rigid structures. Consequently, these structures can conform to the surrounding ground and withstand greater total and differential ground movements in all directions.

- Surface deflections can be controlled by the installation of additional nails or stressing the upper level of nails to a small percentage of their working loads.

Disadvantages

Soil nailing has the following disadvantages, which may be common to other types of cut walls:

- Permanent underground easements may be required.

- The soil face must exhibit sufficient stand up time to permit construction at each level.

- For construction below the groundwater table, the installation of drainage systems may be difficult to construct, and their long-term effectiveness is difficult to ensure.

- In urban areas, the closely spaced array of reinforcements may interfere with nearby utilities. In addition, horizontal displacements may be somewhat greater than with pre-stressed tiebacks, which may cause distortions to immediately adjoining structures.

- Nail capacity may not be economically developed in highly plastic cohesive soils subject to creep, even at relatively low load levels or where the drill hole would cave.

- The long-term performance of shotcrete facings has not been fully demonstrated, particularly in cold climate areas subject to freeze-thaw cycles.

2.3 FEASIBILITY EVALUATION

The feasibility of using soil nailing is generally developed by considering a number of factors namely

- geometric constraints
- soil and groundwater conditions
- disadvantages
- displacements
- cost considerations

The effects of these factors are discussed below.

Geometric Constraints

The required length of nails for construction can vary between 60 – 140% of the wall height, but more typically, from 70 – 90% of the wall height. Therefore, sufficient right-of-way must be available for construction. In urban areas, utilities may interfere with the top row of nails, which are typically installed from 0.5 – 1 meter (1.6 – 3 ft.) below the top of wall. Utility location can be a significant constraint for successful implementation.

Soil and Groundwater Conditions

Based on our current knowledge, soil nailing should not be considered or may not be economical in the following soil profiles:

- soils containing organic matter and characterized by low strengths
- in soils where insufficient stand up time is available for construction
- cinder, ash, or slag fills
- rubble fills or industrial wastes
- acid mine wastes
- In cohesive soils with liquid limit (LL) greater than 50 and with plasticity index (PI) greater than 20 conditions must be carefully assessed for creep susceptibility. Experience from tiebacks suggests that soils with a consistency index (I_c) of less than 0.9 may be susceptible to creep. These soils should not be considered for permanent soil nailed structures without extensive investigations of their creep potential and consideration in design given to use working loads well below the critical creep load.

The consistency index I_c is defined as

$$I_c = \frac{W_l - W}{W_l - W_p}$$

where:

I_c = Consistency Index
W_l = Liquid Limit
W = Natural Water Content
W_p = Plastic Limit

- In cohesionless soils of uniform size, (D60/D10) less than 2, unless found to be very dense. Nailing may be considered in this case if cut face stabilization is achieved prior to excavation by grouting or slurry wall construction or during excavation through use of temporary face stabilization berms.

- In cohesionless soils, current practice precludes the use of ground anchors when the soils are in a loose state (SPT-N < 4). This lower limit appears prudent at present with nailed structures. Similarly, tiebacks are seldom installed in cohesive soils with unconfined compressive strengths of less than 50 kPa (1000 psf). Similar limitations for nailed structures should be considered.

- Below permanent groundwater table, unless a complete drainage system is installed.

Displacements

Horizontal and vertical displacements should be anticipated in soil nailed structures. Generally, the horizontal displacement at the wall face equals the vertical settlement at the top of the structure. The lateral extent of this zone of vertical movement is limited to a horizontal distance equal to 1 to 1.5 times the height of the structure. Quantitatively, these displacements can be roughly estimated as a function of wall height and soil type, as shown in Table 1.

Designs based on lower global factors of safety (FS < 1.35) (see section 4.3) or with steep nail inclinations (>20°) may develop deflections greater than the range indicated in Table 1.

Table 1. Estimated structure deflection.

	Weathered Rock	**Sand**	**Clays**
Vertical or horizontal deflection at top of wall face 4H/1000	$\frac{H}{1000}$	$\frac{2H}{1000}$	$\frac{4H}{1000}$

Cost Considerations

Costs for soil nailed structures are a function of many factors, including ground conditions, site accessibility, wall size, facing type, level of corrosion protection, temporary or permanent construction, and regional availability of contractors skilled in the construction of soil nailed walls.

For temporary construction as part of a feasibility assessment, a cost range of $170/m^2 – $400/m^2 ($16 – $37/ft.^{2}) can be assumed. For permanent construction with a precast or cast-in-place concrete spacing, a cost range of $400/m^2 – $600/m^2 ($37 – $56/ft.^{2}) can be assumed. In general, these costs are on the order of 15% less than alternative anchored walls. Higher costs should be anticipated for small projects in remote areas. A summary of project costs is presented in Chapter 7.

2.4 LIMITATIONS

For soil nail wall construction to be economical, the ground must stand unsupported on a vertical or sloped cut of 1 – 2 m (3 – 7 ft.) for one to two days, to allow nail and temporary shotcrete construction and drill holes to stay open without casing.

Soils considered favorable to soil nailing are weathered rock; natural cohesive materials (silts and low plasticity clays that are not prone to creep); naturally cemented or dense sands and gravels; and fine to medium homogeneous sand with capillary cohesion of at least 3 – 5 kPa (63 – 104 psf) associated with a moisture content of at least 5 – 6%. (There are sometimes face stability problems with this last soil type, particularly for south-facing cuts subjected to drying by the sun.) Soil nailing is also very adaptable from a construction viewpoint and is, therefore, appropriate for mixed-face conditions, such as competent soil over rock.

Soil nailing is not recommended in ground with water pressure present at the face, or in clean or very loose coarse sands and gravels that are either un-cemented or without capillary cohesion, or in organic or clay soils with a LL > 50. In addition, clay soils with a PI > 20 must be carefully assessed for long-term creep, as well as expansive swelling soils. Questionable applications include sites in rock profiles with weak structural discontinuities dipping adversely toward the face or in highly frost susceptible soils in cold climates, unless a frost protection barrier is provided.

CHAPTER 3
CONSTRUCTION SEQUENCE AND MATERIALS

3.1 DESCRIPTION

Soil nailing typically consists of a reinforced shotcrete facing constructed incrementally from the top down and an array of inclusions (nails) drilled and grouted or driven into the soil mass. Prefabricated concrete panels or cast-in-place concrete can be subsequently constructed in front or on the shotcrete facing, if aesthetic or durability considerations warrant the additional expense.

More specifically, the construction for permanent walls generally proceeds as follows:

1. A cut in the soil is made to a depth governed by the ability of the soil to stand unsupported, but no greater than the required vertical spacing of the nails.

2. Prefabricated vertical drainage strips and/or horizontal drains are installed, as required.

3. The nails are installed at predetermined locations to a specified length and inclination, using a drilling and grouting method appropriate for the soil in which they are constructed.

4. Immediately after nail installation, the freshly exposed surface is covered with a reinforced shotcrete layer.

5. Steps 3 and 4 may be inverted, only if short-term face stability is questionable.

6. The nails may be pre-stressed to a small percentage of their working loads, against a small plate secured on the initial shotcrete layer. The pre-stress load usually does not exceed 20% of the working load.

7. A second layer of reinforced shotcrete may be applied, as required. Vertical drainage must be extended downward with each excavation lift prior to shotcreting.

8. The process is repeated for all subsequent levels.

9. Drainage is collected at the base and suitably outletted.

10. For permanent walls, a cast-in-place, prefabricated concrete facing or other type facing is suitably connected to the completed shotcreted structure.

The complete construction sequence is illustrated in Figures 2 through 7.

3.2 EQUIPMENT AND METHODS

There are four major activities connected with soil nailed wall construction:

- excavation
- drilling and grouting nails
- construction of a shotcrete face, which may be temporary
- construction of a final permanent facing

Each activity requires different equipment and methods. Excavation, nail installation and grouting are discussed below.

a. Excavation

Excavation for the face is performed by conventional excavation equipment. Care must be taken to produce as smooth a cut face as possible and not to over-excavate. Irregular or jagged excavation faces make reinforcing mesh steel placement difficult and contribute to considerable overruns in shotcrete quantities. In weathered rock profiles, line drilling prior to excavation should be considered to ensure a relatively smooth face surface.

b. Nail Installation

Current installation methods are classified as

- grouted nails
- self-drilling nails
- jet grouted nails and self-drilling nails (temporary only)
- helical (temporary only)
- driven nails (temporary only)

Figure 2. Excavation of First Lift, and Placement of Drainage Strips.

Figure 3. Drilling a Nail Hole.

Figure 4. Shotcreting of Face.

Figure 5. Grouting a Nail Hole.

Figure 6. Steel Reinforcement Placement Prior to Second Stage Shotcrete Layer at Facing.

Figure 7. Construction of Cast-in-place Final Facing.

Grouted nails are the most common method and are suitable for both temporary and permanent construction. They are placed in boreholes that are advanced by rotary drilling, percussion drilling, auger drilling, or driven casing. Grouting is performed by gravity or under low pressure from the bottom of the drill hole. Spacing typically varies from 1.2 – 1.8 m (4 – 6 ft.) on center. Drill-hole diameter will vary from 100 mm (4 in.) with rotary drilling to 300 mm (12 in.) when using augers.

Most soil nailing is installed using small hydraulic, track-mounted drill rigs. These rigs are usually of the rotary/percussive type that use sectional augers or drill rods. The rigs can work off benches as small as 5 meters wide, but are more productive if benches are 7 meters or more. In dense soils, air tracks have been successfully used.

Large hydraulic-powered track-mounted rigs with continuous flight augers have occasionally been used to install nails up to 28 m (90 ft.) in length. These rigs have the advantage that they can drill the entire length of the nail in a single pass without having to add sectional augers. Their main disadvantages are that they have a large mobilization cost and require a much wider work bench than the smaller, more common, drill rigs.

Open hole drilling is the most economical method of installing drilled-in soil nails. Other drilling techniques used in caving soils include percussive methods, which displace soil by driving casing with a knock-off point on the end, and rotary-percussive methods, which displace soil by drilling and driving drill rods. These nail holes typically have diameters of 90 – 115 mm (3.5 – 4.3 in.). Other cased hole methods of drilling are also used to install soil nails. ***These more expensive drilling methods may increase the cost of soil nail walls significantly to the point where alternative wall construction methods, such as tieback walls, may be more economical.***

Cased methods of drilling include the single tube rotary method, which involves drilling with a single tube (drill string) and flushing the cuttings outside the tube by air, water, or a combination of water and air. The duplex rotary method is also used and is similar to the single tube rotary method, except an outer casing is used that allows drill cuttings to be removed through the annular space between the inner and outer casing. Cased drill hole sizes are generally 90 – 140 mm (3.5 – 5.5 in.) in diameter.

Driven nails are suitable for temporary construction and have been #8 to #11 steel bars or structural angles for greater driving rigidity. They are closely spaced at 2 to 4 nails per 1 m^2 (10.7 $ft.^2$), creating a homogenous composite reinforced soil mass. The nails are driven using vibro-percussive pneumatic or hydraulic top hammers. This installation technique is rapid and economical (3 to 5 nails per hour), but is limited in the length of nail installed by equipment considerations and further limited to soil conditions in which boulders and coarse gravel, weathered rock or cemented soils are absent. This technique has been widely used in France and once in the United States in Provo Canyon, Utah. A similar proprietary technique using explosives for driving steel nails using an air gun was developed in the United Kingdom and was recently demonstrated in the United States. It appears to be particularly well suited for small slope stabilization projects. Driven nails can be considered for permanent installations in non-aggressive ground, using oversized nails providing sacrificial steel and provided that all nails be proof tested.

Jet grouted nails are of a composite construction made from a grouted soil with a central steel nail installed simultaneously. They may be used for temporary applications only. Nails can be installed using vibro-percussion driving at high frequencies (up to 70 Hz) and extremely high grouting pressures (13,000 kPa (1900psi)).[2] The grout under this technique is injected through a small-diameter longitudinal channel in the nail or through a thin steel tube welded to the nail, under a pressure sufficiently high to cause hydraulic fracturing of the surrounding soil. This technique is covered by a European patent and has been little used in North America to date. Alternately, significantly lower grouting pressures (1,400 kPa (200 psi)) have been used in practice with a variety of self-drilled nails. These nails consist of hollow bars that can be drilled and grouted in one operation. In this technique, the grout is injected through the hollow bar simultaneously with the drilling. The grout, which exits through ports located in a sacrificial bit, fills the annulus from the top to the bottom of the drill-hole. Rotary percussive drilling techniques are used with this method. The soil nail type allows for a faster installation than for drilled and grouted nails, and provides some level of corrosion protection. Jet grouting techniques provide, in addition, re-compaction and improvement of the surrounding soil and can increase, significantly in granular soils, the shear and pullout resistance of the soil. This system is most commonly used for temporary construction, and the method is not currently approved for FHWA projects.

c. Grouting

For open hole drilling, gravity tremie grouting is normally used. To increase capacity, second stage grouting has been used, which is accomplished by installing the nail with a re-grout pipe attached. Under this technique, after initial grouting, additional grout is pumped through the re-grout pipe under pressure, fracturing the initial grout, thereby creating a better bond between the soil and grout. The initial grout is normally a neat cement grout with a water-cement ratio of about 0.4 – 0.5 by weight, achieving a compressive strength of 21,000 kPa (3000 psi). Re-grouting should not be used except locally, because it increases costs significantly and requires specialized expertise.

3.3 MATERIALS OF CONSTRUCTION

a. Nails

Ductile steel bars with yield strengths in the range of 420 – 520 N/mm^2 (61 – 75 ksi), with or without corrosion protection, are typically used for conventional drill and grout soil nails. Typical diameters range from 19 mm – 38 mm (0.75 in. – 1.5 in.). High-strength bars commonly used for post-tensioned tiebacks should be avoided because of concerns about more brittle behavior under bending moments.

The most common corrosion protection system consists of either epoxy coating (12 mil minimum) or full grout encapsulation.

Encapsulated corrosion protected nails are used for permanent structures requiring a high degree of corrosion protection. Encapsulation can be achieved by inserting the nail in a plastic corrugated sheath or steel tube and filling the annulus with grout prior to or during grouting of the drill hole. Certain encapsulation techniques may be proprietary.

Centralizers are placed over the entire length of the nail assembly on approximately 2.5 m (8 ft.) spacing (3 m (10 ft.) maximum) to ensure adequate grout cover over the full length of nail.

b. Facing

The facing functions to ensure local ground stability between reinforcements, limit decompression immediately after excavation, and protect the retained soil from surface erosion and weathering effects. The type of facing controls the aesthetics of the structure, as it is the only visible part of the completed work. Depending on the application, the following facings have been used:

- welded wire mesh
- shotcrete
- precast concrete
- cast-in-place concrete

Welded wire mesh can be used for both temporary and permanent applications. It is used in weathered rock profiles or strongly cemented bouldery soils, where surface erosion is not considered to be significant. For permanent applications, galvanization is usually required.

Shotcrete facings are used for both temporary and permanent structures. Shotcrete provides a continuous, flexible surface layer that can fill voids and cracks on the excavated face. For permanent applications, it is always reinforced with a welded wire mesh or rebar cage, with the required thickness obtained by successive layers of shotcrete, each 50 – 100 mm (2.0 – 3.9 in.) in thickness.

Temporary applications have been constructed using both welded wire reinforcement or fiber reinforcement. Shotcrete suitable for facings has been produced by either the dry mix or wet mix process. Dry mix and pneumatic feed wet mix shotcrete use a stiff mixture (water-cement ratio of about 0.4) producing roughly the same quality, although wet mix process shotcrete yields a slightly greater flexural strength. The durability of shotcrete is enhanced by keeping water cement ratios to about 0.4 and using air entrainment, which is extremely difficult with dry mix processes.

A brief comparison of the processes is given in Table 2.[(4)]

Table 2. Comparison of operational features of dry mix & wet mix shotcrete processes.

Dry Mix	Wet Mix
Mixing water and consistency of mix are controlled at nozzle.	Mixing water controlled at delivery equipment and can be accurately measured.
Better suited for mixes containing lightweight porous aggregates.	Better assurance that the mixing water is thoroughly mixed with other ingredients. This may result in less rebound and waste.
Capable of longer hose lengths.	Less dust accompanies the gunning operation.

Steel fiber reinforcement has been added to shotcrete in the wet process to increase ductility, toughness, and impact resistance. Fibers tend to reduce crack propagation but have little effect on compressive strength and produce only modest increase in flexural strength.

Precast concrete facing has been used in permanent applications to provide a finished product to meet a variety of aesthetic, environmental, and durability criteria. They also provide a means of integrating a continuous drainage blanket behind the facing and a frost-protection barrier in cold climates. The prefabricated panels can be attached to the nails or nail head assembly by a variety of devices. In Europe, the connection has often been made at the corner of each large precast panel by using truncated wedging heads between adjoining panels at each nail location. Alternately, nails can be attached to vertical pre-fabricated or cast-in-place columns and panels inserted between columns, such as lagging is inserted in soldier pile and lagging walls. These connection details require a high degree of precision in locating nails, and templates are used to ensure accuracy. It should be noted that specific connection details developed by contractors may be covered by European or American patents.

In the United States, precast concrete panels have been connected to nail head assemblies by rods that are inserted in horizontal slots cast in panels, allowing for horizontal tolerances. Vertical connecting slots in assemblies connected at the nail head obtain vertical tolerances. Although successfully constructed on a number of projects, this connection detail is prone to excessive bending when alignment is poor and must be protected from corrosion.

Alternately, the nail plate may be provided with shear stud connectors, as well as hooped re-bars cast at the back of the precast panels. The space between them is then filled with the low-strength concrete to form the connection. This method has been used with some success in the Eastern United States. A project using this method in Virginia is shown in Figure 8.

Cast-in-place facings are often used for permanent applications. The connection between the nail head assembly and the concrete can be made by extending the nail and bolting an additional plate to provide the required anchorage in the concrete. Alternately, stud connectors are welded to the nail plate assembly to provide for the connection. This latter method is more commonly used.

(*Courtesy JTE Inc.*)

Figure 8. Concrete Being Placed Between the Precast Panels and the Rough Shotcrete Face to Make the Connection.

CHAPTER 4
DESIGN OF SOIL NAILED STRUCTURES

4.1 FUNDAMENTAL CONCEPTS

Although there is at present a wide divergence of design methods used among practitioners in the United States and in Europe, there is general agreement that designs must consider the following potential modes of failure in developing length, spacing, and size of nails:

- internal failure
- mixed or compound failure
- external failure

Schematically, these modes are illustrated in Figure 9.

The basic concept underlying the design of soil nailed structures relies on the transfer of tensile forces generated in the nails in an active zone to a resistant zone through friction (or adhesion) mobilized at the soil nail interface. Some minor passive resistance also develops on the surface perpendicular to the direction of soil-nail relative movement.

The frictional interaction between the ground and the nails restrains ground movement during and after construction. The resisting tensile forces mobilized in the nails induce an apparent increase of normal stresses along potential sliding surfaces (or rock joints), increasing the overall shear resistance of the native ground. The chief design concern is to ensure that soil nail interaction is effectively mobilized to restrain ground displacements and ensure structural stability with an appropriate factor of safety.

The construction of a soil nailed mass results in a composite coherent mass similar to reinforced fill systems (MSE). The locus of maximum tensile forces separates the nailed soil mass in two zones:

- **an active zone (or potential sliding soil or rock wedge)**, where lateral shear stresses are mobilized and result in an increase of the tension force in the nail.

- **a resistant (or stable) zone**, where the generated nail forces are transferred into the ground, as shown in Figure 10.

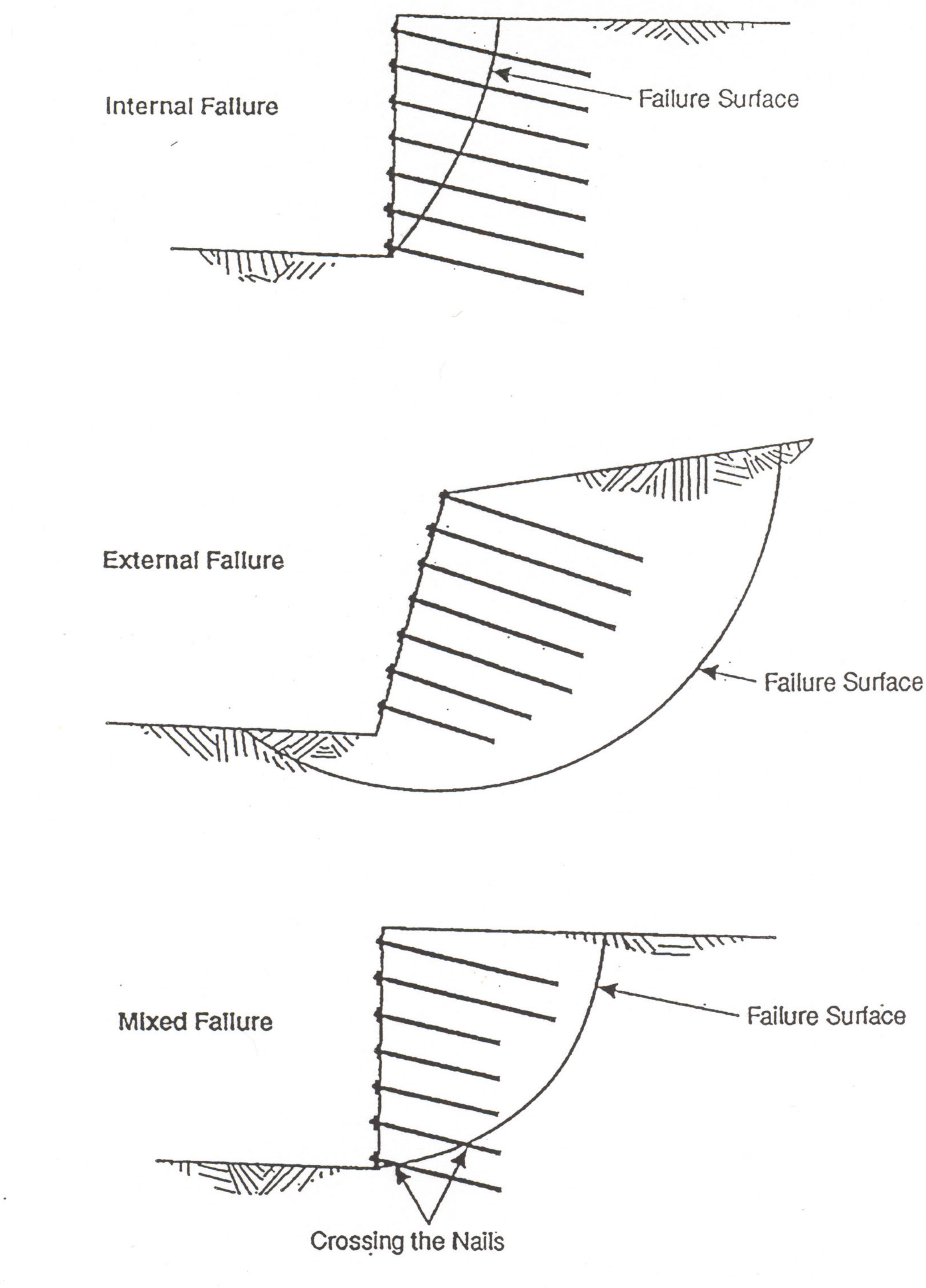


Figure 9. Different Types of Failure Surfaces to be Analyzed for Soil Nailed Walls.

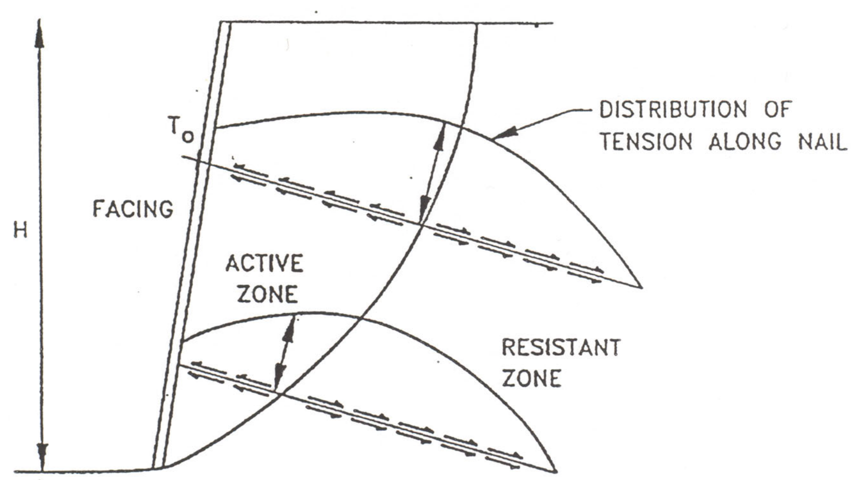


Figure 10. Conceptual Soil Nail Behavior.

The soil nail interaction is mobilized during construction, and displacements occur as the resisting forces are progressively mobilized in the nails.

Most of the widely used design methods to date are based on limit equilibrium design concepts and examine the stability of free body blocks, defined by failure slip surfaces of circular, log spiral, or bi-linear shape.

As in traditional slope stability analyses, limit equilibrium conditions are used to search for the most critical failure surface, which is the failure surface with the lowest factor of safety. Most approaches consider only the tensile capacity of the nails as an addition to the shear resistance of the soil that is mobilized to prevent movement of the soil mass. A few can consider, in addition, the effects of shear capacity and bending stiffness of the nails on the overall structure stiffness and/or the structural capacity of the facing. One method uses a fundamentally different approach for local stability, and kinematically develops the location of the failure surfaces and values of tensile and shear forces both at working and limit stress conditions.

All of the outlined methods have been successfully used to design soil nailed structures worldwide. They are summarized in Table 3, with respect to essential features.

Table 3. Soil nail design methods.
(after Elias et al.[8])

Method	Analysis Type	Failure Surface	Failure Mechanism	Output	Current Usage	Computer Code Availability
1. German Method (1979)	Limit Force Equilibrium, Global Stability	Bi-linear	Pull-out	Global F.S. Critical Failure Surface	Germany	Bauer Program, proprietary
2. Davis Method (1981) Modified 1988-1990.	Limit Force Equilibrium, Global Stability	Parabolic	Pull-out or nail yield stress	Global F.S. Critical Failure Surface	U.S.	In public domain. Modified codes by various contractors.
3. French Method TALREN (1983)	Limit Moment Equilibrium, Global Stability	Circular, or of any shape	Pull-out or nail yield stress	Global F.S. Critical Failure Surface	France	TALREN, proprietary
4. Kinematical (1988)	Working Stress Analysis, Local Stability	Log Spiral	Pull-out or nail yield stress	Nail Forces, Critical Internal Failure Surface	U.S.	In public domain from Polytechnic U.
5. Caltrans-SNAIL (1991)	Limit Force Equilibrium, Global Stability	Bi-linear	Pull-out, nail yield stress, punching shear	Global F.S. Critical Failure Surface, Average Nail Stress	California (U.S.)	SNAIL, in public domain Version 5.01 currently available.
6. Golder Assoc. GOLDNAIL (1991)	Limit Force and Moment Equilibrium, Global Stability	Circular	Pull-out, nail yield stress. Facing shear flexure.	Global F.S. Critical Failure Surface, Considers influence of facing.	U.S.	GOLDNAIL, proprietary

Limit equilibrium methods that consider the structural role of the facing by defining the nail head strength at each level, based on limiting flexure and punching shear capacity of the facing, are typically used in the United States. The details, features, and limitations of the commonly used computer programs based on these concepts, SNAIL and GOLDNAIL, are outlined in FHWA Geotechnical Engineering Circular # 7, FHWA-IF-03-017.[1] The analysis of soil nail walls must consider both the "during construction" and "post-construction" loading conditions to establish the most critical case at all nail levels.

Soil nail spacing in the horizontal and vertical direction must be such that each nail has an influence area $\leq 4\ m^2$ ($\leq 40\ ft.^2$) and minimum soil nail spacing of 1.0 m (3.3 ft.). The minimum length of nail should be 0.5 H (height of wall). For gravity grouting and efficient nail tensile capacity, a minimum nail inclination of 15° should be considered.

4.2 SOIL-NAIL INTERACTION

During excavation, due to lateral decompression of the soil, nails are loaded primarily in tension. The transfer of stresses, between the soil and nails is primarily accomplished through skin friction up to the ultimate capacity of the soil. The ultimate resistance of the nail is, therefore, a function of the perimeter area of the grouted nail and the nature and density or shear strength of the soil.

At present, there is a general consensus that there is no viable theoretical relationship that can accurately predict nail pullout capacity.

Preliminary designs are, therefore, based on field correlation studies and experience. This imposes a strict requirement for field testing during construction to verify design assumptions and modify the design where needed.

The ultimate unit frictional resistance for grouted nails, q_u (bond strength) values based on data from FHWA IF-03-017, and pressuremeter correlations in Europe should be considered for preliminary design evaluation. They are summarized in Table 4.

Table 4. Estimated bond strength of soil nails in soil and rock.[1]

Material	Construction Method	Soil/Rock Type	Ultimate Bond Strength, q_u (kPa)
Rock	Rotary Drilled	Marl/limestone	300 - 400
		Phyllite	100 - 300
		Chalk	500 - 600
		Soft dolomite	400 - 600
		Fissured dolomite	600 - 1000
		Weathered sandstone	200 - 300
		Weathered shale	100 - 150
		Weathered schist	100 - 175
		Basalt	500 - 600
		Slate/Hard shale	300 - 400
Cohesionless Soils	Rotary Drilled	Sand/gravel	100 - 180
		Silty sand	100 - 150
		Silt	60 - 75
		Piedmont residual	40 - 120
		Fine colluvium	75 - 150
	Driven Casing	Sand/gravel	
		low overburden	190 - 240
		high overburden	280 - 430
		Dense moraine	380 - 480
		Colluvium	100 - 180
	Augered	Silty sand fill	20 - 40
		Silty fine sand	55 - 90
		Silty clayey sand	60 - 140
	Jet Grouted	Sand	380
		Sand/gravel	700
Fine-Grained Soils	Rotary Drilled	Silty clay	35 - 50
	Driven Casing	Clayey silt	90 - 140
	Augered	Loess	25 - 75
		Soft clay	20 - 30
		Stiff clay	40 - 60
		Stiff clayey silt	40 - 100
		Calcareous sandy clay	90 - 140

NOTES: Convert values in kPa to psf by multiplying by 20.9.
Convert values in kPa to psi by multiplying by 0.145.

4.3 DESIGN OF STRUCTURES

Current practice consists of developing nail length, size, and spacing based on methods outlined in Section 4.1. Details of these methodologies are beyond the scope of this Technical Summary and are outlined in FHWA-IF-03-017.[1]

a. Factors of Safety

Designs based on limit equilibrium calculate a factor of safety defined as the ratio of the resisting forces and/or moments to the driving forces and/or moments. Where a single global factor of safety is used, it has been common practice to apply it to shear strength and to use factors of 1.35 for "non-critical" and 1.5 for "critical structures" under static loading. To date, Service Load Design (SLD) methods have been used almost exclusively.

The design method outlined in FHWA-IF-03-017 consists of selecting a trial nail size, length, and spacing, determining the allowable nail load by considering both the allowable tensile stress and factored pullout resistance, and then calculating the global limiting equilibrium factor of safety for the critical slip surface. The limiting equilibrium calculations take into account the allowable nail load and the ultimate soil strength.

A minimum global factor of safety of 1.35 is recommended for non-critical structures, and 1.5 for critical structures with the design procedures developed, as well as a factor of safety for pullout capacity of 2.0 and the current AASHTO criteria for tensile yield of steel.

b. Soil Strength Parameters for Design

The success and economy of design is predicated on a reasonably accurate determination of the soil parameters used in limit equilibrium analyses, which form the basis for design. For cohesionless soils, the shear strength is represented by a drained effective angle of internal friction. The value is commonly estimated from field tests (e.g., SPT and CPT) summarized in Table 5.

For fine-grained soils, both drained and undrained strengths are necessary to evaluate stability. Undrained strength is consistent with construction and end-of-construction stability, and drained strength consistent with long-term stability.

Table 5. Correlations between SPT and CPT and friction angle of cohesionless soils.[1]

	In-Situ Test Results	Relative Density	ϕ' (degrees) (a) [3]	ϕ' (degrees) (b) [4]
SPT N-Value [1] (blows/300 mm or blows/ft.)	0 to 4	Very Loose	< 28	< 30
	4 to 10	Loose	28 to 30	30 to 35
	10 to 30	Medium	30 to 36	35 to 40
	30 to 50	Dense	36 to 41	40 to 45
	> 50	Very Dense	> 41	> 45
Normalized CPT cone bearing resistance (q_c/P_a) [2], [4]	< 20	Very Loose	< 30	
	20 to 40	Loose	30 to 35	
	40 to 120	Medium	35 to 40	
	120 to 200	Dense	40 to 45	
	> 200	Very Dense	> 45	

NOTES:
1. *SPT N-values are field, uncorrected values.*
2. *P_a is the normal atmospheric pressure = 1 atm ~ 100 kN/m^2 ~ 1 tsf.*
3. *Ranges in column (a) from Peck, Hanson, and Thornburn (1974).*
4. *Ranges in column (b) and for CPT are from Meyerhof (1956).*

Undrained strength is commonly estimated from unconfined compression tests (UC) or unconsolidated undrained triaxial tests (UU). Drained strength can be estimated from drained triaxial compression tests or consolidated undrained triaxial tests with pore pressure measurements.

The bond developed between the grouted nail and soil can be estimated from the correlations presented in Table 4.

c. Face Stability

Local stability of the soil face during excavation is one of the most important considerations in soil nail wall selection and construction. This failure mode is not amenable to conventional stability analysis and is typically addressed during the design and wall selection phase by a field test cut to demonstrate that the face can stand unsupported for sufficient time to allow nail and construction facing installation. Local sloughing of the face, possibly extending through to the surface, can be relatively sudden and is most prevalent at shallow depths where loose fill/highly weathered material is more likely to be encountered.

d. Preliminary Design for Feasibility Evaluations

For preliminary design and feasibility evaluations to determine nail lengths and sizes, with simple geometry and homogeneous soils, design charts based solely on the major parameters can be developed based on any limit equilibrium method. Design charts based on methods outlined in FHWA-IF-03-017 and using the SNAIL computer program are included in this section as an example. These charts have been prepared for a common nail inclination of 15 degrees, homogeneous ground conditions, and a global FS of 1.35. (For FS values other than 1.35, apply a correction factor – see Figure 15). The charts shown as Figures 11 – 15 are presented in dimensionless format with the following variables:

Backslope Angle, β - Design charts are presented corresponding to backslope angles of 0 or 10 degrees. For intermediate backslope angles, interpolate between the charts.

Face or Batter Angle, α - For each backslope angle, design information is presented for two face or batter angles of 0 and 10 degrees from the vertical.

Nail Inclination i – Constant at 15 degrees.

Strength Variables - Friction Angle, ϕ, and normalized cohesion C* defined as the cohesion normalized with respect to the total soil unit weight and the vertical height of the wall (H).

$$C^* = \frac{c}{\gamma H}$$

Maximum Design Nail Force – T_{max-s} - The maximum design nail forces are calculated for the most critical failure surface by correcting the maximum nail forces T_{max} calculated for a global $FS_G > 1$, by multiplying this term by the ratio of <u>average</u> nail forces T_{avg-s}/T_{avg}. T_{avg-s} is the average nail force calculated for $Fs_G = 1.0$, and T_{avg} is the average nail force calculated for $FS_G > 1$. As such, this maximum design nail force takes into account the soil strength full mobilization. The normalized maximum design nail force, t_{max-s}, is then expressed as

$$t_{max-s} = \frac{T_{max-s}}{\gamma H S_H S_V}$$

Normalized Pullout Resistance, μ, is equal to:

$$\mu = \frac{q_u \, D_{DH}}{FS_P \gamma \, S_H \, S_V}$$

where FS_P is the factor of safety against pullout (typically 2.0); D_{DH} is the drill-hole diameter; γ is the total unit weight of the soil behind the wall; and S_H and S_V are the horizontal and vertical nail spacing, respectively. The nail lengths in the charts were computed based on the most critical failure surface (i.e., considering base and toe failures) for the selected geometry and material properties, and assuming that failure of the nail (i.e., tensile breakage) and/or failure of the facing would not take place. Therefore, the pullout failure is implicitly assumed. The charts are further based on a drill-hole diameter of 100 mm (4 in.) and a cohesion intercept, c, such that $c^* = c/\gamma H = 0.02$. For drill-hole diameter or cohesion intercept values different than the assumptions above, adjustments to the calculated nail length and maximum tensile forces are made by means of correction factors (refer to Figure 15).

e. Preliminary Design Procedure

The preliminary design based on chart solutions will provide adequate information for feasibility evaluations and determining ROW requirements and preliminary costs. They may be used as a starting point for detailed design.

The procedure for using the design charts to determine length and size of nails, in conjunction with the dimensionless variables discussed above, consists of the following steps:

1. For a specific project application, evaluate batter (α), backslope (β), effective friction angle (ϕ'), and ultimate bond strength (q_u). Calculate normalized pullout resistance (μ) with assumed or trial vertical and horizontal spacings (S_V and S_H).

2. Obtain normalized length (L/H) from the appropriate chart (Figures 11a to 14a).

3. Obtain normalized force (t_{max-s}) from the appropriate chart (Figures 11b to 14b).

4. Using Figure 15, evaluate correction factors for (a) normalized length to account for a drill-hole diameter other than 100 mm (4 in.) (correction factor C_{1L}), (b) a c* value

other than 0.02 (correction factor C_{2L}), and (c) a global factor of safety other than 1.35 (correction factor C_{3L}).

5. Using Figure 15, evaluate correction factors for normalized maximum nail force to account for (a) a drill-hole diameter other than 100 mm (4 in.) (correction factor C_{1F}), and (b) a c* value other than 0.02 (correction factor C_{2F}).

6. Apply correction factors to normalized length and/or normalized force. Calculation method is provided in Figure 15.

7. Multiply the normalized length by the wall height to obtain the soil nail length.

8. Calculate the maximum design load in the nail T_{max-s} using the value of t_{max-s} from

$$t_{max-s} = \frac{T_{max-s}}{\gamma\ H\ S_H\ S_V}$$

9. Calculate the required cross-sectional area (A_t) of the nail bar according to

$$A_t = \frac{T_{max-s} FS_T}{f_y}$$

   where f_y is the steel yield strength and FS_T is the factor of safety for nail bar tensile strength.

10. Select closest commercially available bar size using Table 6 that has a cross-sectional area of at least that evaluated in the previous step.

11. Verify that selected bar size fits in the drill-hole with a minimum grout cover thickness of 25 mm (1 in.).

12. If the length and/or nail diameter are not feasible, select another nail spacing and/or drill-hole diameter, recalculate the normalized pullout resistance, and start the process again.

f. Preliminary Design Example

Design for the following geometry: 10m wall, α= 0, β=0 and i=15°; a global factor of safety, FS = 2; and following soil properties for a clayey sand: ϕ= 33°, c = 2.5 kPa, γ= 20 kN/m^3.

Based on the above and Table 4, the ultimate adhesion, q_u, is estimated at 100 kPa.

A trial design is made based on D_{DH} = 150 mm, and a nail spacing of 1.5 m both directions for a FS_p = 2.0

$$\mu = \frac{q_u\ D_{DH}}{FS_P \gamma\ S_H\ S_V} = 0.167$$

$$c^* = c/\gamma H = 2.5/20 \times 10 = 0.0125$$

From Figure 11a, L/H = 0.73, and from Figure 11b, $t_{max\text{-}s}$ = 0.18.

Correct for Drill Hole Diameter C_{1L}, Cohesion C_{2L} and Factor of Safety C_{3L} by using Figure 15.

For C_{1L} directly from Figure 15 = 0.83
For C_{2L} = - 4.0 x c* + 1.09 = 1.04
For C_{3L} = 0.52 FS + 0.30 = 1.34

Corrected L/H = 0.83 x 1.04 x 1.34 x 0.73 = 0.84

The maximum tensile force $t_{max\text{-}s}$ also needs to be corrected for the larger drill hole diameter C_{1F} from Figure 15 and the previously calculated soil cohesion C_{2F}.

From Figure 15 C_{1F} = 1.47
C_{2F} = 1.04

The corrected normalized force is

$$t_{max\text{-}s} = C_{IF} \times C_{2F} \times t_{max\text{-}s} = 1.47 \times 1.04 \times 0.18 = 0.28$$

The maximum design nail force at the failure surface is

$$T_{max\text{-}s} = \gamma\ H\ S_H\ S_V\ t_{max\text{-}s} = 20 \times 10 \times 1.5 \times 1.5 \times 0.28 = 126 \text{ kN}$$

The maximum design nail force at the nail head, T_o is

$$T_o = T_{max\text{-}s}\ [\ 0.6 + 0.2\ (S_V\ \{m\} - 1)] = 126\ (0.6 + 0.1) = 88.2 \text{ kN}$$

The nail capacity R_T based on the maximum nail force with a FS of 1.8 is

$$R_T = FS_T\ T_{max\text{-}s} = 1.8 \times 126 = 227 \text{ kN}$$

The required section using 525 MPa steel, therefore, is

$$A_t = R_T / f_y = 227 / 0.525 = 432 \text{ mm}^2$$

From Table 6 a 25-mm size is selected.

In summary, the design yields a length of L = (L/H) H = 0.84 (10) = 8.4 m, say 8.5 m, with 25 mm nails at an equal vertical and horizontal spacing of 1.5 m.

Additional Stability Concepts

External failure modes generally refer to failure modes where the soil mass is treated as a block, the block being the height of the structure with the depth being equal to the length of the nails. On this basis, conventional analyses, as with any gravity structure, must be made to ensure that the block is of sufficient mass to preclude a sliding failure at the base or if founded in soft- to medium-stiff cohesive soils that adequate bearing capacity exists.

Seismic effects on external stability must be considered where the ground accelerations are generally greater than 0.09g.

For details of the necessary analyses and methods refer to Section 5.4.5 of Geotechnical Engineering Circular # 7.[1]

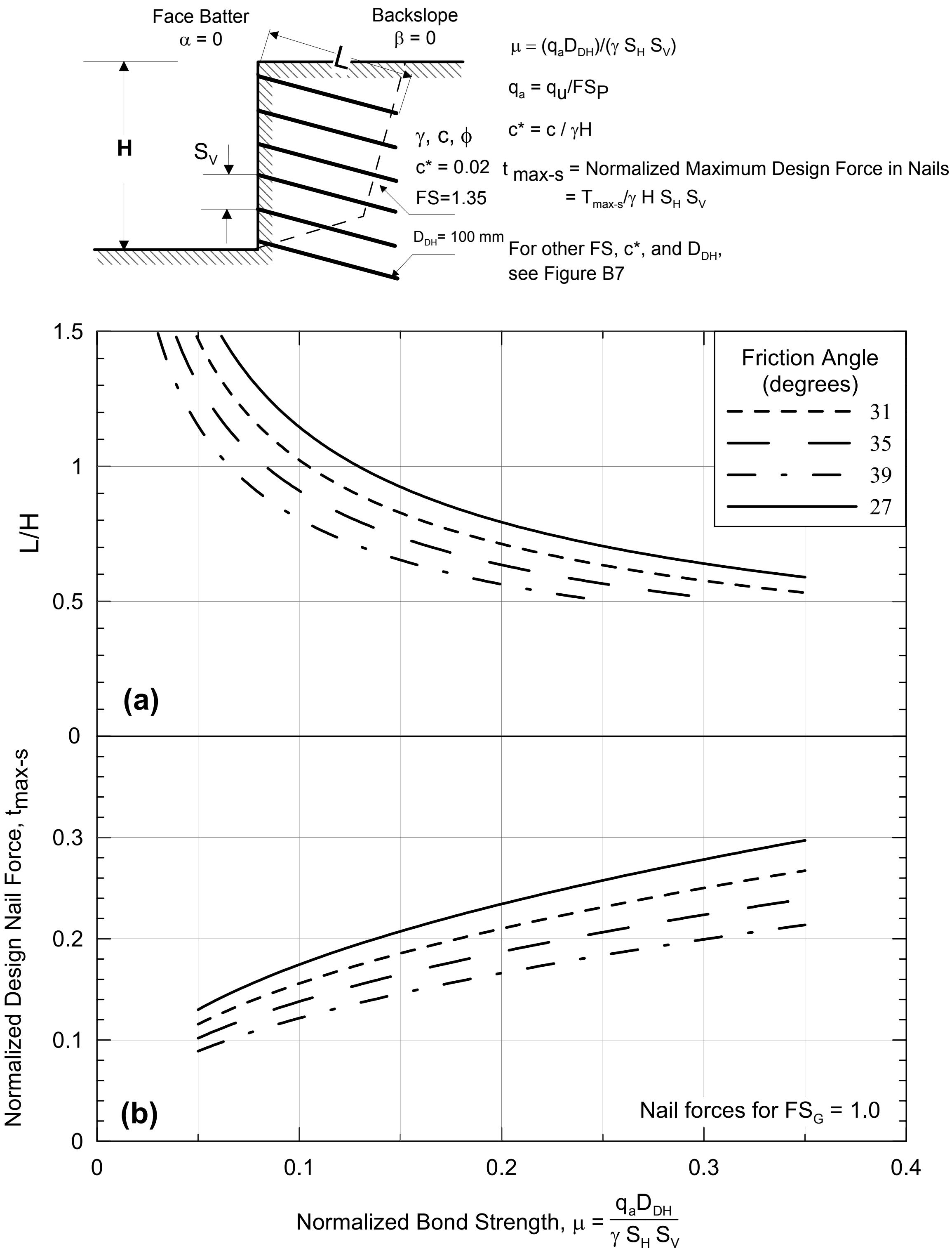


Figure B.1: Batter 0° - Backslope 0°

Figure 11. Design Chart: Face Batter 0°, Backslope = 0°. (1)

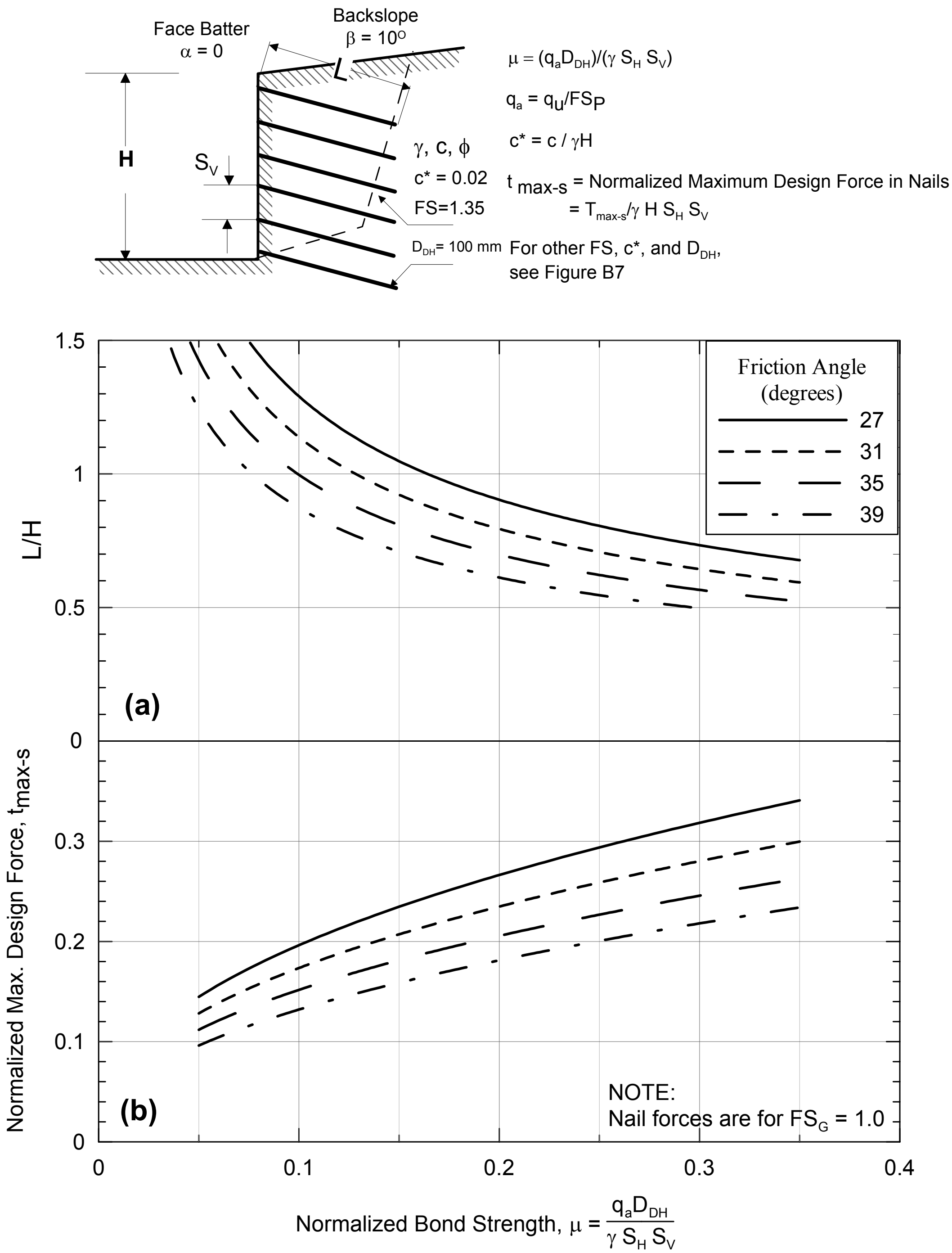


Figure B.2: Batter 0° - Backslope 10°

Figure 12. Design Chart: Face Batter = 0°, Backslope = 10°.(1)

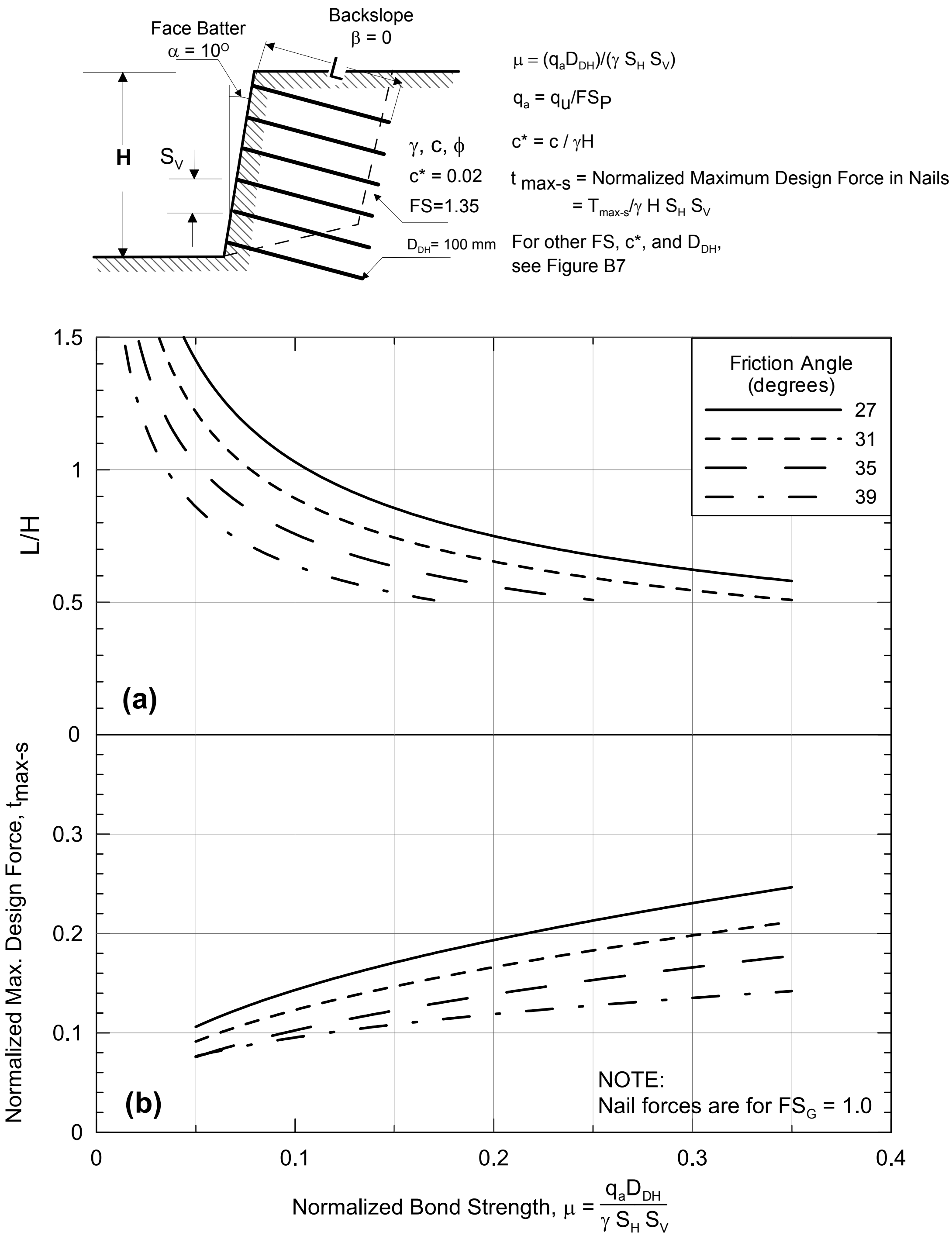


Figure B.3: Batter 10° - Backslope 0°

Figure 13. Design Chart: Face Batter = 10°, Backslope = 0°. (1)

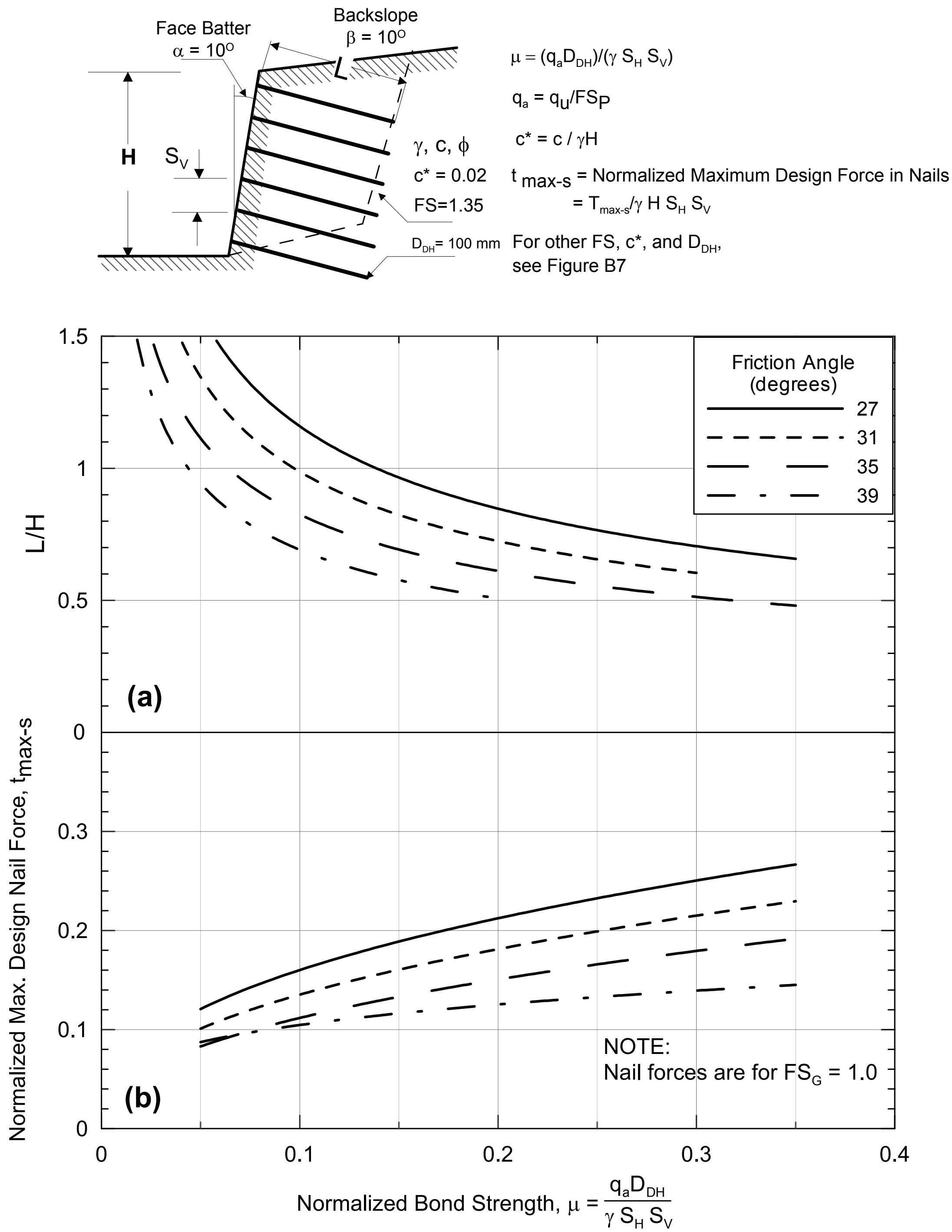


Figure B.4: Batter 10° - Backslope 10°

Figure 14. Design Chart: Face Batter 10°, Backslope 10°.(1)

Corrections of Soil Length

$$\frac{L}{H}\text{ (corrected)} = C_{1L} \times C_{2L} \times C_{3L} \times \frac{L}{H}\text{ (from charts for } D_{DH} = 100\text{ mm, } c^* = 0.02\text{ , } FS_G = 1.35\text{)}$$

where:

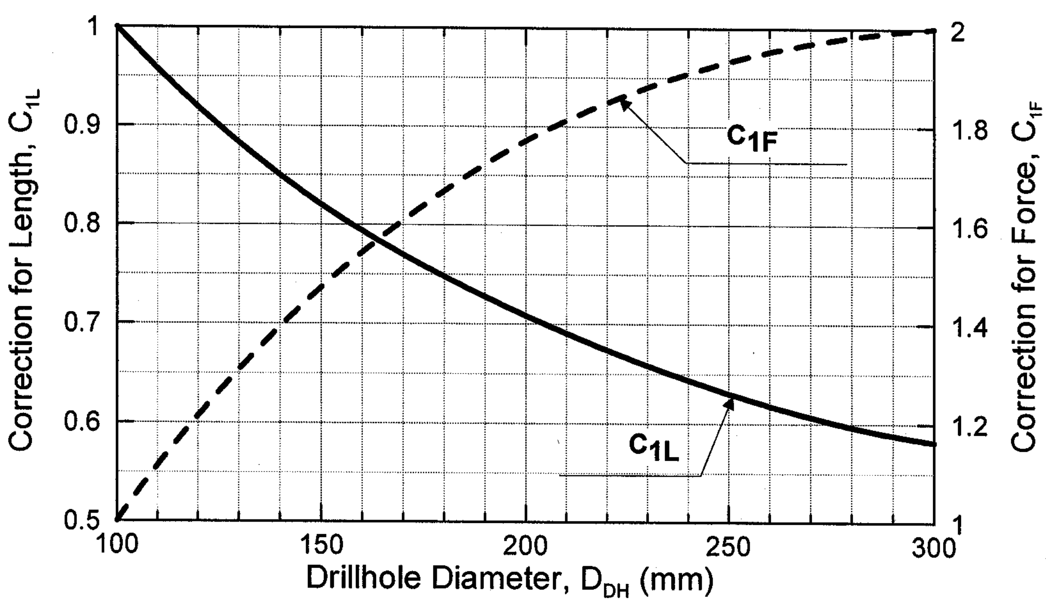
C_{1L} = Correction for Drillhole Diameter — see chart (a) below
C_{2L} = Correction for Soil Cohesion — see formula (b) below
C_{3L} = Correction for Global Factor of Safety — see formula (c) below

Corrections of Normalized Maximum In-Service Nail Force

$$t_{max\text{-}s}\text{ (corrected)} = C_{1F} \times C_{2F} \times t_{max\text{-}s}\text{ (from charts for } D_{DH} = 100\text{ mm, } c^* = 0.02\text{)}$$

C_{1F}= Correction for Drillhole Diameter — see chart (a)
C_{2F}= Correction for Soil Cohesion — see formula (b)

(a) Correction for Drillhole Diameter



(b) Correction for different values of normalized cohesion ($c^*=c/\gamma H$)

$C_{2L} = -4.0\ c^* + 1.09 \geq 0.85$

$C_{2F} = -4.0\ c^* + 1.09 \geq 0.85$

(c) Correction for different global factors of safety

$C_{3L} = 0.52\ FS + 0.30 \geq 1.0$

Figure 15. Design Chart for Correction Factors. [(1)]

Table 6. Threaded bar properties.[1]

Nominal Bar Designation		Cross-Sectional Area		Nominal Unit Weight		Max. Diameter w/Threads		ASTM Grade	Yield Strength		Max. Axial Load	
English	mm	in.^{2}	mm^2	lbs/ft.	kg/m	in.	mm	English	ksi	MPa	kips	kN
#6	19	0.44	284	1.50	2.24	0.86	21.8	60	60	414	26.4	118
								75	75	517	33.0	147
#7	22	0.60	387	2.04	3.04	0.99	25.1	60	60	414	36.0	160
								75	75	517	45.0	200
#8	25	0.79	510	2.67	3.98	1.12	28.4	60	60	414	47.4	211
								75	75	517	59.3	264
#9	29	1.00	645	3.40	5.06	1.26	32.0	60	60	414	60.0	267
								75	75	517	75.0	334
#10	32	1.27	819	4.30	6.41	1.43	36.3	60	60	414	76.2	339
								75	75	517	95.3	424
#11	36	1.56	1,006	5.31	7.91	1.61	40.9	60	60	414	93.6	417
								75	75	517	117.0	520
#14	43	2.25	1,452	7.65	11.39	1.86	47.2	60	60	414	135.0	601
								75	75	517	168.8	751

g. Facing Design

For permanent walls, one of the following options is generally chosen.

- **Permanent Exposed Shotcrete Facing.** Present technology for shotcrete placement is such that the final shotcrete layer can be controlled to close tolerances and, with nominal hand finishing, an appearance similar to a CIP wall can be obtained (if desired). The shotcrete, whether left in the natural gun finish or hand textured, can also be colored, either by adding coloring agent to the mix or by applying a pigmented sealer or stain over the shotcrete surface. The finished total thickness is generally between 180 – 250 mm (7 – 10 in.).

- **Separate Fascia Wall (CIP or Precast Panels).** The temporary shotcrete finish can be covered with a separate fascia wall consisting either of a CIP wall or precast face panels. The CIP section is typically a minimum 200 mm (8 in.) thick for constructability, and shear stud connectors are welded to the nail bearing plates to transfer load. A typical face design is shown in Figure 16.

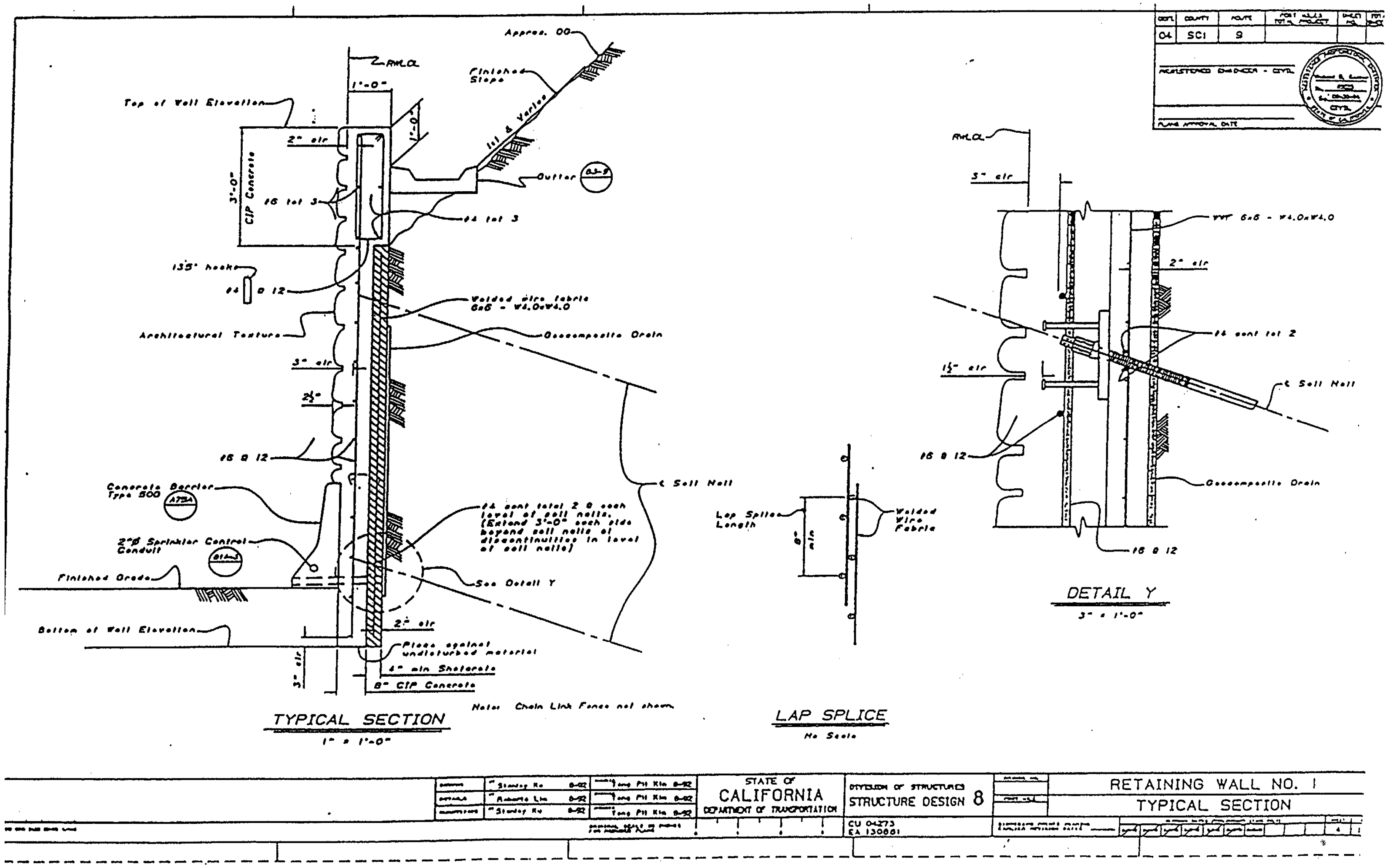


Figure 16. CALTRANS Details.

Precast face panels can be smaller modular panels or full-height fascia panels, such as those used to cover permanent slurry walls. A disadvantage of the smaller modular face panels is difficulty of attaching the face panels to the nail heads and proprietary restrictions on certain connection details. A disadvantage of full-height precast panels is that due to constructability, weight, and handling limitations, their use is often limited to wall heights less than 8 m (26 ft.).

Facings have been designed by purely empirical methods based on experience, but presently they are designed by modeling the facing as a continuous two-way slab/raft on an elastic foundation supported by the nails.

The nail forces at the facing have been computed by either considering the maximum tensile force that can be carried by each nail or by empirical relationships based on measured nail tensile forces at the facing, which demonstrated a reduction of the maximum nail tensile load at the face as a function of nail spacing. The latter is recommended, and detailed design guidance is given in Geotechnical Engineering Circular # 7.[1]

A preliminary permanent facing design can be obtained by considering the following potential failure modes:

- Flexural Failure
- Punching Head Failure
- Headed Stud Tensile Failure

Facing Resistance R_{FF}

As with any reinforced concrete structure, the reinforcement in the facing must fall within proscribed limits that are a function of the compressive strength of the concrete and the tensile strength of the steel reinforcement. For the usual concrete compressive strength of 21 MPa and steel tensile strength of 420 MPa, the reinforcement ratio falls between 0.22 – 1.5%.

The adequacy of an initially assumed or typical reinforcement can then be ascertained. Referring to the previously designed 10 m wall, a typical 200 mm (h) concrete facing with 13 mm bar @ 300 mm c.c. each way with an area 129mm^2/300mm would be considered typical. The steel area, a_s, per unit length (meter) is

$$a_s = 129 \times 1000/300 = 430 \text{ mm}^2/\text{m}$$

Therefore, the steel ratio ρ, in each direction is

$$\rho = a_s / h/2 \text{ x } 100 = 430 / 1000 \text{ x } 100 \text{ x } 100 = 0.43\%$$

And the total steel ratio is

$$\rho_T = 0.43 \text{ x } 2 = 0.86\%$$

From Table 7, for a 200 mm facing thickness, nail spacing ratio of 1, and a steel ratio of 0.86%, the Flexural Capacity R_{FF} can be interpolated as 270 kN. Considering the previous design example, the 10 m wall, the maximum design nail force was determined as 130.5 kN and the nail head design force as 91.4 kN. Therefore, for structural adequacy, the Flexural Capacity must be at least 1.5 (FS) times the nail head force, which can be expressed as

$$FS_F \; T_o < R_{FF}$$
$$1.5 \text{ x } 91.4 = 137 \text{ kN} < 270 \text{ kN}$$

Therefore the reinforcement chosen is adequate

Facing Punching Shear Resistance R_{FP}

The resistance is a function of the stud height, spacing, and concrete compressive strength. The capacity has to be in excess of the nail head force, plus a FS of 1.5.

$$FS_{PP} \; T_o < R_{FF}$$
$$1.5 \text{ x } 91.4 = 137 \text{ kN} < 270 \text{ kN}$$

From Table 8, and with a concrete compressive strength of 21 Mpa, a spacing of 125 mm with a stud having an effective length of 125mm would satisfy the minimum requirements.

Facing Head Stud Resistance R_{HT}

The tensile capacity of the headed studs providing anchorage to the permanent facing is the last structural design issue. The nail head capacity against tensile failure, $R_{HT,}$ is computed as

$$FS_{FP} \; T_o < R_{HT}$$
$$2.0 \text{ x } 91.4 = 183 \text{ kN} < R_{HT}$$

From Table 9, a stud diameter of 12.7 mm satisfies the above relationship.

Table 7. Facing resistance for flexure, R_{FF} (1).

$h^{(a)}$	Nail Spacing Ratio$^{(b)}$	ρ_{tot} (%) $^{(c)}$		
		0.5	1.0	2.0
mm (in.)	[-]	R_{FF} in kN (kip)		
100 (4)	0.67	50 (12)	105 (24)	210 (48)
	1	75 (18)	155 (36)	315 (71)
150 (6)	0.67	90 (20)	175 (40)	355 (81)
	1	130 (30)	265 (60)	535 (120)
200 (8)	0.67	105 (24)	210 (48)	425 (95)
	1	155 (36)	315 (71)	635 (143)

Based on a reinforcement yield strength, f_y, of 420 MPa (Grade 60). For f_y = 520 MPa (Grade 75), multiply the values in the table by 1.24. For permanent facing, for h = 100 mm (4 in.), divide R_{FF} by 2; for h = 150 mm (6 in.), divide R_{FF} by 1.5; for h = 200 mm (8 in.), use same R_{FF}.

NOTES:
- *(a) h is the facing thickness.*
- *(b) Nail space ratio is the lowest of either S_V/S_H or S_H/S_V, resulting in a value less than or equal to 1.*
- *(c) ρ_{tot} is the total reinforcement ratio calculated as $\rho_{tot} = \rho_n + \rho_m$ where ρ_n and ρ_m are the nail head and mid-span reinforcement ratios, respectively. $\rho_I = a_{ij}/0.5h$, where a_{ij} = cross-sectional area of reinforcement per unit width in "I" direction (vertical or horizontal) and at location "j" (nail head or mid-span).*

Table 8. Facing resistance for shear punching, R_{FP} (1).

$h_c^{(a)}$	$F'_c{}^{(b)}$	Headed-Stud Spacing, S_{HS} (mm/in.)		
		100/4	125/5	150/6
(mm/in.)	(MPa)	R_{FP} in kN (kip)		
100/4	21	95 (21)	95 (21)	95 (21)
	28	110 (25)	110 (25)	110 (25)
125/5	21	130 (30)	145 (33)	145 (33)
	28	150 (35)	170 (39)	170 (39)
150/6	21	175 (40)	195 (44)	210 (48)
	28	205 (46)	225 (51)	245 (55)

NOTES:
- *(a) $h_c = L_s - t_H + t_P$ where: L_s is the effective headed-stud length; t_P is the bearing plate thickness [typically 19 mm (0.75 in.)]; t_H is the headed-stud head thickness.*
- *(b) F_c' is the concrete nominal compressive strength.*

Table 9. Facing resistance for stud, tensile failure, R_{FH} [(1)].

Headed-Stud Shaft Diameter, D_S		R_{FH}
mm	In.	kN (kip)
9.7	3/8	120 (28)
12.7	1/2	210 (48)
15.9	5/8	330 (75)
19.1	3/4	480 (108)
22.2	7/8	650 (146)

Based on 4 headed-studs and a yield strength, fy, of 420 MPa (Grade 60). For f_y = 520 MPa (Grade 75), multiply the values in the table by 1.

h. Corrosion Protection

Corrosion protection for soil nails is based on tieback practice. For *permanent* soil nailed structures, it should consist of

1. A minimum grout cover of 25 mm (1.0 in.) to be achieved throughout the grout zone for nails that are not fully encapsulated. Centralizers should be placed at distances of 2.5 m (8 ft.) center to center, with the lowest centralizers a maximum of 0.3 m (1 ft.) from the bottom of the grouted drill hole.

2. In non-aggressive ground, the nail section could be fusion-bonded epoxy using an electrostatic process to provide a minimum epoxy coating thickness of 0.3 mm (12 mils) in accordance with AASHTO M-284. A minimum grout cover of 25 mm (1.0 in.) is required throughout the length of nail.

3. In aggressive ground or for critical structures (e.g., walls adjacent to high-volume traffic roadways or walls in front of bridge abutments) or where field observations have indicated corrosion of existing similar structures, fully encapsulated nails should be used. Full encapsulation is generally accomplished as with ground anchors, by grouting the nail inside a corrugated plastic sheath. This tube must be capable of withstanding deformations associated with transportation, installation, and passive stressing of the nail. The annular space between the corrugated tube and tendon is

usually filled with a neat cement grout containing admixtures to control bleed of water from grout. Under this procedure the outermost grout cover between the tube and the drill hole wall can be reduced to 12 mm (0.5 in.), and the nail need not be protected by an additional coating.

Critical values that define "aggressive" ground are as follows:

Test	**Critical Value**	**Test Method**
pH	Below 5	AASHTO T-289
Resistivity	Below 2,000 ohm-cm	AASHTO T-288
Sulfate	Above 200 ppm	AASHTO T-290
Chloride	Above 100 ppm	AASHTO T-291

The above tests should routinely be conducted on representative soil samples as part of the subsurface investigation for permanent soil nailed wall applications.

For *temporary* applications in non-aggressive ground, the grout cover of 25 mm (1.0 in.) will provide adequate protection. Centralizers must be provided. In aggressive ground, full encapsulation should be considered.

i. Frost Forces

The formation of ice lenses in the vicinity of the soil nail wall facing in frost-susceptible soils should be anticipated. This has led to the development of high loads on both the facing and the head of the nail, because of the fully bonded nature of the nail and its inability to tolerate large strains in the adjacent soil without developing correspondingly high loads in the nail. This phenomenon has resulted in damage to the facing. In situations where the facing is very resistant, damage can occur to either the nail or to the connection between the nail and the facing.

The magnitude of the facing/nail loads developed will depend on the depth of frost penetration, the intensity and duration of the freeze period, the availability of water and the R value (insulation capacity) of the facing system. Increases in nail and facing loads should be anticipated in areas where frost durations as measured by the Freezing Index in C-days, is in excess of 300 C-days and where there are frost-susceptible soils near the face, as well as in close proximity to a source of water.[6] The Freezing Index is the summation of freezing-degree days over a freezing season and are, therefore, a measure of the average daily temperature below freezing.

Frost-susceptible soils are generally characterized by having more than 3% finer than the 0.02 mm size. However, for frost pressures to develop, both frost-susceptible soils and water availability must be present. Water sources may be perched or groundwater levels above the bottom of the wall or from infiltration from surface runoff.

Frost loading effects may be eliminated or mitigated by the use of porous backfill between the construction shotcrete facing and a precast facing (used on a few projects to date), or by placing insulating material either between the shotcrete construction facing and the CIP or precast panel final facing, or by placing insulating material outside the permanent concrete facing.

Alternately, frost forces may be estimated based on the data presented by Kingsbury.(6)

j. Wall Drainage Systems

Since walls are not normally designed to resist hydrostatic forces, surface and/or subsurface drainage should be provided by one of the following methods:

- **Prefabricated Drains:** Minimum 300-mm-wide geocomposite drains can be placed vertically prior to applying shotcrete. Typical center-to-center spacing is the same as the horizontal nail spacing. The drain mats are extended down the full height of the excavation. At the base, they discharge into a collector pipe, suitably outletted.

- **Surface Interceptor Ditch:** A shallow ditch behind the top of the wall to lead away surface water. Ditch lining can be shotcrete or cast-in-place concrete.

- **Weep Holes and Horizontal Drains:** Fifty-millimeter-diameter PVC pipe weep holes placed on approximate 3 m (10 ft.) centers can be installed through the shotcrete face. These are plugged temporarily when shotcrete is applied. Longer 50-mm (2.0 in.) PVC horizontal drain pipes can also be installed in heavy seepage areas.

CHAPTER 5
BIDDING METHODS, CONSTRUCTION CONTROL AND CONSTRUCTION SPECIFICATIONS

5.1 INTRODUCTION

Two general types of contracting methods for soil nailed walls can be considered:

- Method or Procedural Specifications is one option, with all details of design, construction materials and all construction methods, including drilling specified in the contracting documents. ***This is not recommended.***

A variant to this method, as is often used in anchor wall contracting, would allow the contractor to choose nail installation methods required to achieve specified nail capacities, while specifying minimum and or maximum requirements for diameter and length. This variant has been widely used.

- Performance or end result approach, is the second option, with lines and grades noted on the drawings with design criteria and performance requirements specified. Under this method, a project-specific review and detailed plan submittal occurs in conjunction with normal working drawings submittal.

Some user agencies prefer one approach to the other or a mixed use of approaches, based upon criticality of a particular structure. Both contracting approaches are valid if properly implemented and each has advantages and disadvantages.

This chapter outlines the necessary elements, the requirements imposed on the owner, the construction control requirements, and the current construction specifications.

5.2 METHOD OR PROCEDURAL SPECIFICATIONS

This contracting approach includes the development of a detailed set of plans and material specifications in the bidding documents.

The advantage of this approach is that the complete design, details, and material specifications can be developed and reviewed over a much longer design period. This approach further empowers agency engineers to examine more options during design but requires an engineering staff trained in this technology. This trained staff is a valuable asset during construction, when questions and/or design modifications are required.

Under this contracting procedure, the agency is fully responsible for the design and performance of the soil nail system, as long as the contractor has installed the components (nails, facing, drainage) in accordance with the specifications. The agency assumes all risks and is responsible for directing the work, if changes to the design are indicated or warranted.

The use of a variant to this method, in which the contractor is responsible for developing the required nail capacity by varying the drilling and grouting methods, drill hole diameter, and length of nails from specified minimums, has several advantages. It empowers the contractor to use his experience and specialized equipment to the best advantage, and allows the agency to share the major risk, nail capacity for a specified length, with the contractor.

To implement this contractual method, the following additional information must be included in a special provision:

- the results of the geotechnical investigations, including all laboratory test results
- submittal requirements for the contractor, outlining drilling and grouting methods
- minimum drill hole diameters and length
- required soil nail design capacity at each level or location

This type of contracting procedure typically results in a better or more economical end product. Permanent soil nailed walls are often contracted for in this manner.

The use of a method specifications contracting is recommended only for agencies that have developed sufficient in-house expertise and consider soil nail wall design and construction control as conventional or standard method for earth retention.

5.3 PERFORMANCE OR END RESULT SPECIFICATIONS

Under this approach, often called *line and grade* or *conceptual plans*, the agency prepares drawings for the geometric requirements for the structure, material specifications for the components, determines performance requirements, and indicates the range of acceptable construction methods. The end result approach with sound specifications and pre-qualification of contractors offers several benefits. Design of the structure is performed by trained and experienced staff and can utilize the contractor's proprietary equipment and methods. The material components (facing, nails, and miscellaneous) have been successfully and routinely used together, which may not be the case for in-house design with generic specifications for components. Also, the end result specification approach lessens engineering costs and manpower for an agency, and transfers some of the project's design cost to construction.

The disadvantage is that agency engineers must have adequate expertise in soil nailing to perform a design review and approve construction modifications, or it must engage a consultant with demonstrated proficiency in this technology.

The bid quantities are obtained from specified pay limits denoted on the "line and grade" drawings and can be bid on a lump sum or unit price basis per meter square. The basis for detailed designs to be submitted after contract award is contained as complete special provisions, as would construction control and monitoring requirements.

The plans furnished, as part of the contract documents, should contain the geometric, geotechnical, and design-specific information listed below.

- Plan and elevation of the areas to be retained, including beginning and end stations.
- Typical cross-section that indicates face batter, pay limits, drainage requirements, excavation limits, etc.
- Elevation view of each structure, showing original ground line, minimum foundation level, finished grade at ground surface and top of wall or slope line.
- Location of utilities, signs, etc., and the loads imposed by each such appurtenance, if any.
- Construction constraints, such as staged construction, right-of-way, construction easements, etc.
- Magnitude, location, and direction of external loads due to bridges, overhead signs and lights, and traffic surcharges.
- Limits and requirements of drainage features beneath, behind, above, or through the structure.
- Reference to specific governing sections of an Agency Design Manual (Materials, Structural, Hydraulic, and Geotechnical), construction specifications, and special provisions. The Agency may specify as part of the special provisions, acceptable design methods by referencing one or more of the methods outlined in this Technical Summary, or qualifying methods that the contractor uses as part of its pre-qualification process.

- Provide or make available the results of the geotechnical investigation, but preferably the interpreted soil design parameters.
- Specify submittals that the contractor must provide, including calculations, drawings, and construction methods.
- Specify design safety factors, material properties, and requirements.
- Specify the level of corrosion protection required.
- Specify the finished face requirements.
- Specify wall alignment tolerances and allowable horizontal movements.
- Specify the percentage of nails to be tested, testing procedures, and acceptance criteria.
- Establish wall construction monitoring requirements.

Performance or end result specifications are indicated for agencies with limited experience with this technology or for complicated projects where a specialty contractor's specific or local knowledge can be used to the best advantage.

5.4 CONSTRUCTION MONITORING

Prior to construction of the soil nailed structure, personnel responsible for construction control and monitoring should become familiar with the following items

- Plans, specifications, and testing requirements.
- Site conditions relevant to construction conditions.
- Material requirements and allowable tolerances.
- Construction sequencing.
- Pre-qualification requirements for specialty contractors and required data in compliance with this requirement.

Quality assurance measures must be implemented to

- Inspect steel nails for damage and availability of mill test certificates to certify grade and corrosion protection, where required.
- Maintain stability of excavated face at all stages. If stability cannot be maintained at the initial depth of cut, smaller depths with immediate subsequent shotcreting are required.
- Install the nails at the correct orientation spacing and length. In drilling the nail hole, the contractor must maintain an open hole without any loss of ground, or casing must be used. Drilling muds are not recommended due to probable loss of capacity from the bentonite residue on the hole perimeter. Drilling operations must be such as to

prevent loss of ground. Subsidence of ground above the drilling location or large quantities of soil removal with little or no advancement of the drill head should not be permitted.

- Ensure proper location of the nails in the drilled hole by the use of centralizers. Insertion of the nail may be done before or after tremie grouting the drill hole. The nail must be inserted to its prescribed length. Inability to achieve this penetration in uncased holes usually means caving of the soil into the hole and requires re-drilling. Centralizers must be of such design as not to impede the flow of grout in the borehole.
- Proper grouting of the borehole around the nail. The grouting operation involves injecting grout at the lowest point of the drill hole in order to fill the hole evenly without air voids.
- Assure adequate shotcrete strength, thickness, and proper placement of reinforcement in the shotcrete facing. Collect grout tubes and shotcrete cores for strength testing.
- Maintain proper placement of the bearing plate. Deviations of perpendicularity between the plate and nail should be adjusted by using tapered washers below the nut. The plate must be fitted with small holes to allow for grouting and return flow that ensures no void exists between the primary nail grout surface and the bearing plate.
- Maintain proper installation of deep drains, weepholes, and prefabricated vertical drains. It is essential that hydraulic continuity of the vertical drain is assured, if installed incrementally.

Nail Testing

Nail testing in each representative soil strata is an extension of design and is used to verify or establish design criteria. Specific requirements are outlined in the construction specifications. The grouted test length should be not less than 2.5 m (8 ft.), and a non-grouted zone of at least 1 m (3 ft.) should be provided behind the face of the structure. Tests are conducted using constant load techniques, and carried out preferably to pullout failure or to the design pullout capacity times the required factor of safety. Detailed requirements for inspection and QA/QC testing are fully outlined in FHWA SA-93-068 "Soil Nailing; Field Inspectors Manual."(3)

5.5 CONSTRUCTION SPECIFICATIONS FOR PERMANENT SOIL NAILED WALLS

The recommended construction specifications, updated in 2003, are fully developed in FHWA-IF-03-017.(1)

CHAPTER 6
COST DATA

6.1 PERMANENT STRUCTURES

Table 10 summarizes historical bid data compiled under FHWA-SA-96-069.

Table 10. U.S. highway cost data.

Project	Year	Wall m^2	Facing Type	Bid/m^2
I-78 Allentown, PA	1987	645	SP	$580
C.G. Tunnel, KY	1988	1,000	S	$390
I-10 San Bernardino, CA	1988	810	S	$290
I-5 Tacoma, WA	1989	185	S	$430
I-495 G-W Parkway, VA	1990	1,358	CIP	$580
I-35 Laredo, TX	1990	205	CIP	$340
RT 23A Hunter, NY	1990	777	CIP/STONE	$748
I-5 Portland, OR	1990	382	S	$630
RT 37 Vallejo, CA	1990	1,620	CIP/PP	$510
RT 85 San Jose, CA	1990	4,438	CIP	$300
I-66 over I-495 Fairfax, VA	1990	330	CIP	$1,000
RT 85 San Jose, CA	1991	8,909	CIP	$330
Hwy 101 San Jose, CA	1991	3,234	CIP	$390
I-880 Industrial Parkway, Hayward, CA	1991	169	CIP	$520
Hwy 50 Sacramento, CA	1991	257	CIP	$450
RT 89 Tahoe Pines, CA	1991	604	CIP	$420
R 400, Atlanta, GA	1991	467	CIP	$746
RT 85 San Jose, CA	1992	4,591	CIP	$270

Table 10 (cont.)

Project	Year	Wall m^2	Facing Type	Bid/m^2
RT 85 Saratoga, CA	1992	1,732	CIP	$380
RT 85 San Jose, CA	1992	3,434	CIP	$230
RT 680 Walnut Creek, CA	1992	1,863	CIP	$300
Interstate 80, Berkely, CA	1992	314	CIP	$495
Tovawanda Dr., San Diego, CA	1992	975	CIP	$422
RT 121 Napa Co., CA	1992	301	CIP	$536
I-80, Elmsford Park, NJ	1993	316	CIP	$1,242
RT 28 Fairfax, VA	1993	6,080	CIP	$360
RT 2 Dixon, IL	1993	446	CIP	$431
RT 167 Seattle, WA	1993	552	CIP	$336
RT 101 Olympia, WA	1993	1,429	CIP	$393
I-80 Olympia, WA	1993	1,414	CIP	$163
I-5 Tukwila, WA	1993	407	S	$718
I-5 Seattle, WA	1993	3,277	CIP	$326
RT 217 Portland, OR	1993	1,958	CIP	$411
RT 217 Portland, OR	1993	164	CIP	$419
RT 26 Portland, OR	1993	1,425	CIP	$410
I-5 Seattle, WA	1994	102	CIP/TIMBER	$645
I-70 St. Louis, MO	1994	231	S/TEMP	$459
I-40 Greensboro, NC	1994	400	CIP	$777
RT 50 Cannon City, CO	1994	102	S/TIMBER	$645
I-35 Pensall, TX	1995	539	CIP	$393

NOTES: 1. Facing Type Key - CIP - cast-in-place; S - shotcrete; SP - shotcrete, precast panels

2. $m^2 = 10.76\ ft^2$

Careful review of the data indicates that where the soil nailed walls with CIP facings were bid competitively, and not submitted as value engineering alternates or no unusually complicated architectural treatment was required, the average bid price was on the order of $380/m^2 (\$35/ft.^{2}). Architectural finishes usually add \$30/m^2 (\$3/ft.^{2}). Precast panel or timber-faced walls averaged \$600/m^2 (\$56/ft.^{2}).

6.2 TEMPORARY WALLS

Less data are available for temporary wall construction, as temporary shoring is seldom a bid item and is usually paid as incidental to other construction. Limited information suggests costs from \$160/m^2 – \$400/m^2 (\$15 – \$37/ft.^{2}).

The difference in cost is based on the cost of the permanent facing (200$^+$ mm (8$^+$ in) of cast-in-place concrete or precast panels) and corrosion protection for nails.

CHAPTER 7
CASE HISTORIES

7.1 SOIL NAILED ABUTMENT I-495, VIRGINIA[7]

Project Description

The underpass widening on I-495 (the Washington, D.C., Capital Beltway) at the George Washington Memorial Parkway (GWMP) involved the excavation of a concrete-protected earthen end slope embankment below the spread footing of the west abutment of the GWMP overpass structures. Figure 17 shows the project location and geometric conditions for construction.

The geometric configuration of I-495 at the GWMP prior to construction was 3 through-lanes of traffic and an exit-only lane for the off-ramp to the GWMP. Excavation of the end slope embankment was required in order to provide the necessary geometry for a new alignment for an off-ramp from I-495 to a 2-lane collector road and 4 through-lanes of traffic on I-495.

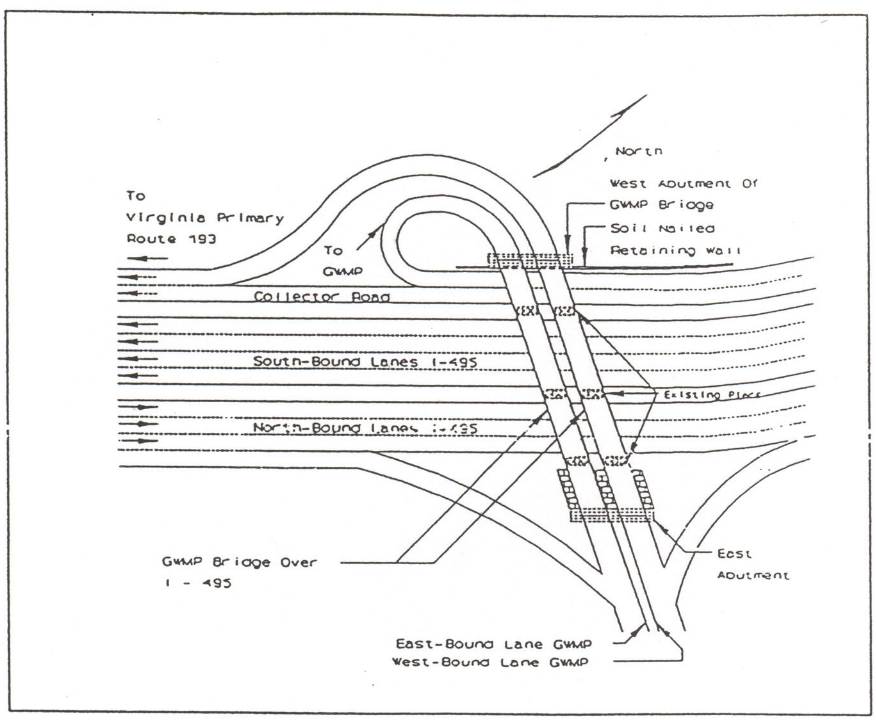


Figure 17. Plan View of Project Location after Construction of Soil Nailed Retaining Wall.

The contract plans required the construction of a permanent steel H-pile and timber lagging anchor retaining wall adjacent to the west abutment of the GWMP bridge to provide for lateral support and stability for the bridge abutment's spread footing foundation. The contractor approached the Virginia Department of Transportation (VDOT) with a proposal for the construction of an alternate permanent soil nailed retaining wall to solve the problem of driving H piles under the bridge's limited available headroom or through the bridge deck and to provide a cost incentive to the project under the value engineering clause.

Because this soil nailed retaining wall was believed to be the first installation to support a bridge abutment in the United States, it was designated as an experimental feature by FHWA, and instrumentation was incorporated to monitor horizontal movements.

Soil Properties

Engineering soil properties at this site were not easily assessed, due to a lack of soil classification tests (Atterberg limits, natural moisture content, and gradation), and assumptions were made to facilitate the design process. A soil unit weight of 18.8 kN/m^3 (120 pcf) was used throughout this design. A design value of $\phi = 28^o$ was used for the angle of internal friction, and c = 0 for the cohesion of the soil. In qualitative terms, the soil was a very dense saprolite composed of a light brown micaceous silt interlaced with weathered schistose fragments. Standard penetration test N values were found to be between 25 and 50. The assumed strength values were, therefore, conservative as extensive reported testing for Piedmont residual soils usually yields ϕ values of 30 to 32 degrees. Consistent with the above, an adhesion of 88 kPa (1840 psf) between the soil and the grout was assumed as a limiting value, based on general available correlations and local knowledge.

Design Development

Plans for the construction of the soil nailed retaining wall required the wall to be a total of 198 meters (650 ft.) in length. Of this, approximately 33 m (110 ft.) were underneath the GWMP bridge abutment. Given this geometry, the soil nailed retaining wall was designed to resist the induced earth pressure and additional abutment surcharge where applicable. For this design, the surcharge loading of the abutment was 250 kPa (5225 psf). Those sections of the wall that were not subject to the abutment surcharge were designed to support a sloped (2 to 1) embankment surcharge behind the wall.

The design was performed by considering both local and global stability, as applicable.

A kinematical limit analysis (FHWA RD89-198) was made to determine minimum nail lengths based on local equilibrium and as a means of determining the additional capacity or lengths required to resist the additional loads due to the abutment footing load. A factor of safety of 1.75 for adhesion was used. The resulting design under the abutment was based on a spacing of 1.5 m (5 ft.) horizontal and 1.22 m (4 ft.) vertical, and a 200-mm-diameter (8 in.) grouted hole. The top 3 rows of nails are longer, with an L/H ratio of 1.0, compared to the bottom 2 rows with an L/H ratio of 0.8.

For the balance of the wall, a spacing of 1.5 m (5 ft.) horizontal and vertical was used with a smaller 130-mm (5 in.) grouted hole. The resulting uniform lengths were consistent with an L/H ratio of 1.3, which proved to be more economical.

The design of the reinforced shotcrete facing for the soil nailed retaining wall was accomplished by considering the facing as a beam of unit width supported by a soil nail at each corner. A shotcrete facing with a thickness of 178 mm (7 in.), which was to be placed in two layers, was designed for this structure. Reinforcement at each soil nail was provided by a 600-mm (23.6 in.) square grid of #4 reinforcement steel secured on 100-mm (4 in.) centers. After the first layer of shotcrete was placed, a 200 mm x 200 mm x 10 mm (8 x 8 x 0.4 in.) steel end bearing plate was then bolted to the end of each soil nail. Reinforcement of the outer side of the facing was provided by W7 welded-wire mesh on 100-mm (3.9 in.) centers. Design details of the shotcrete reinforcement for wall sections supporting the GWMP Bridge abutment and wall sections supporting the 2 to 1 slope were identical.

In addition, a cast-in-place (CIP) concrete fascia was provided as an architectural veneer for aesthetic reasons. The CIP concrete fascia was designed to have a thickness of 150 mm (6 in.), with #4 reinforcement steel placed on 300-mm (12 in.) centers in both the horizontal and vertical directions. As this CIP fascia was merely an architectural veneer and was nonstructural, the reinforcement was only for temperature and shrinkage. A steel plate similar to the end bearing plate was also bolted on the end of each soil nail in the CIP concrete fascia. The purpose of the steel plate was to anchor the CIP fascia to the shotcrete and soil nail. Construction and monitoring details are shown in Figures 18 and 19.

Monitoring

Because of the wall's experimental designation, a monitoring program to verify key design assumptions and monitor horizontal movements was implemented.

Figure 18. Construction of Soil Nailed Retaining Wall under GWMP Bridge Abutment Prior to Shotcreting.

Figure 19. Testing Soil Nails for Capacity.

Six non-service nails were installed and incrementally load tested to either failure or to the ultimate limit adhesion used for design, which required that the design capacity be achieved to a load/deflection of less than 1 mm (0.04 in.). All test nails achieved the required capacity and typically developed a load/deflection behavior similar to the one shown in Figure 20. Two inclinometers were installed on either end of the abutment to monitor horizontal reflections. Post-construction readings indicated horizontal movements on the order of 0.12 – 0.16% of the wall height, which was slightly larger than anticipated for a soil nailed wall in this type of soil.

The cost of construction was $580/m^2$ ($54/ft.^{2}) in 1991. The construction has been maintenance-free to date (2003), and should continue to provide maintenance-free service.

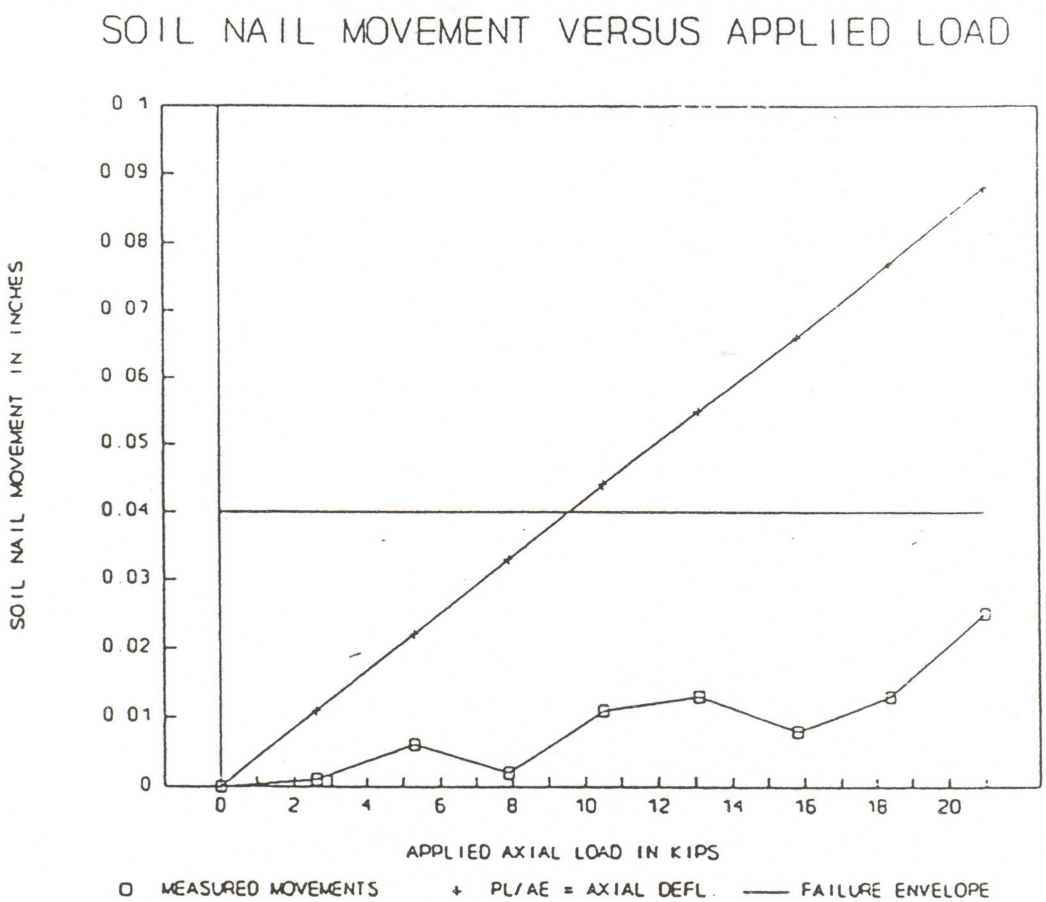


Figure 20. Pullout Test Results.

7.2 SOIL NAILED WALL, TONAWANDA DRIVE, SAN DIEGO, CA.[5]

Project Description

The relocation and improvement to Tonawanda Drive in San Diego, California, required the construction of a retaining wall in a cut section. To fulfill this requirement, a tieback anchor wall was designed, varying in height from 6.4 – 8 m (21 – 26 ft.), supporting a broken back soil surcharge (2:1). The design required the installation of one to two rows of tiebacks at 2.44 m (8.0 ft.) centers, between two W8 x 48 soldier piles encased in a 760-mm-diameter (30 in.) concrete shaft, extending 5.5 – 11 m (18 – 36 ft.) below the lagging level. A cast-in-place architectural concrete face would then be constructed over the lagging.

The contractor approached Caltrans with a proposal for the construction of an alternate permanent soil nailed retaining wall, and provided a considerable cost incentive to the project under a value engineering clause.

Site borings indicated a surficial layer of clayey silty sand overlying the weak silty sandstones of the San Diego Formation or dense cemented sands.

Engineering soil properties were estimated as a soil unit weight of 18 kN/m^3 (114.5 pcf), ϕ of 33^o, cohesion of 24 kPa (502 psf), and soil/grout allowable adhesion of 104 kPa (2174 psf) consistent with a 150-mm (5.9 in.) drilled nail hole diameter.

Design Development

The soil nail plans for the 153 m (500 ft.) long wall were developed based on a horizontal nail spacing of 1.6 m (5.2 ft.). The resulting nail lengths varied from 5.5 – 6.1 m (18 – 20 ft.) or having a L/H ratio of 0.76, with a structural nail cross-sectional area of 25 mm (#8).

The design was performed using the Caltrans SNAIL computer code, utilizing a global factor of safety of 1.5.

A temporary shotcrete face 100-mm thick was estimated, with minimal reinforcement (WWF 6x6, W2xW2) placed in the center, to be finally covered by a 222-mm (8¾-in.) cast-in-place reinforced concrete fascia with an architectural finish. The connection between the facings was provided by four stud connectors welded on each nail bearing plate. Drainage by vertical geocomposite drains was similar in scope and extent to the original design.

Corrosion protection for the nails was provided by resin bond epoxy.

The as-built value engineering wall was bid at $422/m^2$ ($39/ft.^{2}) in 1992, and provided for a savings of $268/m^2 ($25/ft.^{2}).

Field Testing

The major variable in design, the adhesion between the nail grout and the soil, was checked by field pullout testing. The contractor initially elected to drill the nail holes using a 150-mm (6-in.) flight auger. Pullout testing indicated that insufficient capacity was being generated which would mean either longer nails or greater drill hole diameters would be required. The contractor elected to increase the diameter to 200 mm (8 in.), which increased the capacity sufficiently to meet the requirements of the design proposal.

The completed construction is shown in Figure 21.

Figure 21. Completed Tonawanda Wall.

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Technical Summary # 10:

GROUTING

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CHAPTER 1
DESCRIPTION AND HISTORY

From the early days of simple cement slurry injections to today's sophisticated multi-material 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. It must be realized that the field of grouting is extremely large, and continues to grow at an accelerating pace. Long though this chapter is, it constitutes only an introduction, and engineers embarking upon a grouting project are strongly counseled to seek expert advice.

1.1 DESCRIPTION

Grouting comprises a variety of techniques that 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 soils 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 the ground's amenability to different kinds of grout. Integral components of a program are a thorough geotechnical investigation to identify the site conditions and to logically guide the choice of the grouting method and its effectiveness, real time monitoring and analysis of data permitting appropriate adjustments, and a responsive verification program.

1.2 FOCUS AND SCOPE OF MANUAL

Grouting technology continues to advance at a fast pace and indeed this Technical Summary, originally drafted in the late 1990s, should be regarded only as the position of the industry at that time. In the interim, significant advances have been made across the board of grouting techniques, as illustrated in the proceedings of the Conferences held in New Orleans (2003)[66] and Orlando (2004)[67]. With the development of newer techniques, such as compensation grouting, and the progressive refinement of more traditional methods, such as permeation and compaction grouting, grouting now 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 logical basis for strategic 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 that 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, stabilizers, water reducers, and anti-washout agents.

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

Batch – The amount of grout mixed at one time.

Bentonite – Clay mineral, preferably natural sodium montmorillonite. It is used to provide stability to a cement-based grout.

Blaine – The specific surface area of a particulate material, measured in cm^2/g or m^2/kg. Portland cements have a Blaine value of 3,000 – 5,000 cm^2/g. Ultra fine cements have a Blaine value of over 8000 cm^2/g and can reach 11,000 cm^2/g.

Bleeding – Water naturally separating a cement-based grout at rest (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-Based Grout – A suspension mix of cement, water, and various admixtures and additives.

Chemical Grout – A material generally comprising a pure solution, or, in the case of sodium silicates, a natural colloidal solution. Distinct from *Particulate Grout* (below).

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

Colloidal Mixer – A high-speed, high-shear grout mixer that produces a uniform, well hydrated particulate suspension.

Compaction Grouting – Grouting using low mobility and high internal friction, grout (LMG) injected with less than 25 mm (1 in.) 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.

Compensation Grouting – The injection of grout concurrent with underground tunneling to replace lost ground and prevent settlement of structures or the ground at the surface above the tunnel during construction.

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:

$$V = \frac{Q}{A} = K \times \frac{\partial h}{\partial L}$$

where:	V	=	velocity (m/s)
	Q	=	flow rate (m^3/s)
	A	=	cross-sectional area (m^2)
	K	=	coefficient of permeability (m/s)
	$\partial h / \partial L$	=	hydraulic (or pressure) gradient (m/m = 1)

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. Two types are typically used: C and F.

Fracture Grouting – The injection of grout to intentionally fracture the ground hydraulically to create lenses of grout that strengthen ground by reinforcement action and/or produce conrolled heave to lift structures.

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's material or mass after it has set or stiffened.

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

Grouting – The injection under pressure of a fluid into the ground that then solidifies to alter formation's material or mass properties and/or create a structure.

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).

Limited Mobility Grouting – The injection of stiff grout that displaces the soil into which it is injected, does not mix with or permeate the soil, and does not travel far from the point of injection. Compaction grouting is a form of Limited Mobility Grouting.

Mortar – A cement-grout, of low water/cement ratio, 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 grout material comprising 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-micron sieve (i.e., # 200).

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 cementitious material conforming to ASTM C150 with relatively high strength and slow and even setting.

Pozzolan – A siliceous or siliceous-and-aluminous material that possesses little or no cementitious value. In a finely divided form and in the presence of moisture, however, pozzolan reacts chemically with calcium hydroxide to form compounds possessing cementitious properties.

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 objectives.

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. Commonly used to refer to the collective fluid properties of grouts.

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.

Slab Jacking – The injection of grout beneath slabs or shallow foundations with the intent of producing controlled lifting.

Slurry Grout –A fluid mixture comprising solids, such as cement, sand, or clays suspended in water (old term).

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.

Stability (Pressure Filtration) – A measure of the internal stability of a particulate grout when subjected to excess pressure in its fluid state. The higher the amount of water expressed during a standard test, the less stable the grout, and the less attractive it is for injecting into fissures and pore spaces.

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 end of 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 mechanical shearing.

Ultra-Fine Cement – A mixture of finely ground Portland or slag-based cement, often with mineral admixtures. Also known as microfine cement.

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, typically measured in cP (centiPoise).

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, but increasingly measured by volume. Ratio by volume = 1.5 x ratio by weight.

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 "grdt," 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 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 keynote lectures at the International Conference on Grouting[58] were invited to address the historical evolution of the various technologies. The interested reader is referred to the four papers, by Littlejohn (Permeation and Compensation), Lombardi (Rock Fissures), Shibazaki (Jet), and Warner (Compaction). They are truly landmark papers.

The principal types of geotechnical grouting are shown In Figure 1 and are as follows:

1. Rock Grouting
   - Fissures (using High Mobility Grouts (HMG))
   - Voids (natural and artificial, using Low Mobility Grouts (LMG))
2. Soil Grouting
   - Permeation (using HMG and solution grouts)
   - Compaction (or displacement)
   - Jet (or replacement)
   - Fracture (including compensation grouting)

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).

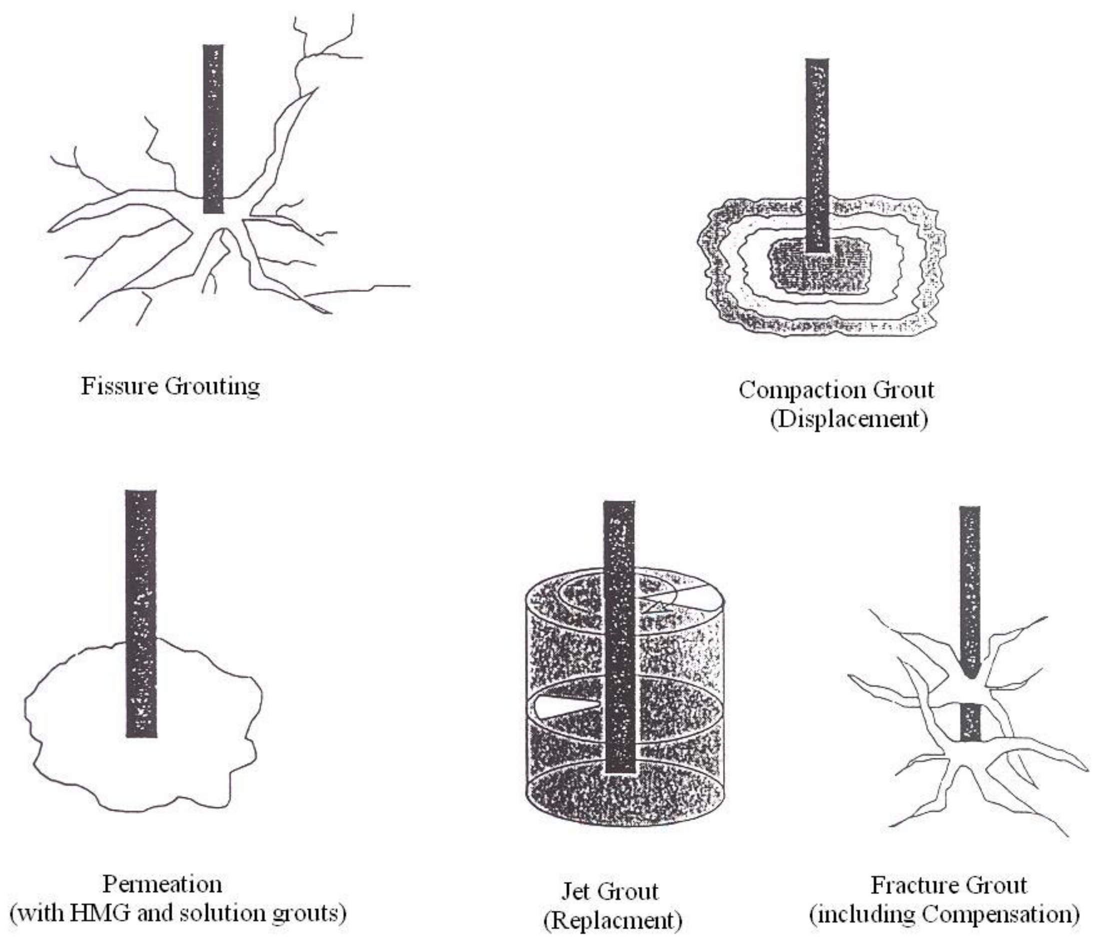


Figure 1. Types of Grouting.

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 port 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, Connecticut, in 1854, and R.L. Harris constructed grouted concrete foundations in 1891 at Croton Lake, New York, 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, New York).[5]

There then followed considerable activity with HMGs 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, Oregon, 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 that 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 in the absence of foreign input. As a result, by the early 1980s, American practice was certainly different from, and arguably somewhat behind, European and Japanese 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 or iron mines). Such voids can generate surface settlements and/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, while more exotic materials, such as hot bitumen,[59] are necessary to stop high flow/high head flows into quarries, and/or under dams.

As a subset of void-filling operations, the term "slabjacking" (or "mudjacking") is common, as illustrated in Figure 2, although it is not a rock grouting technique or application. 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.

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]

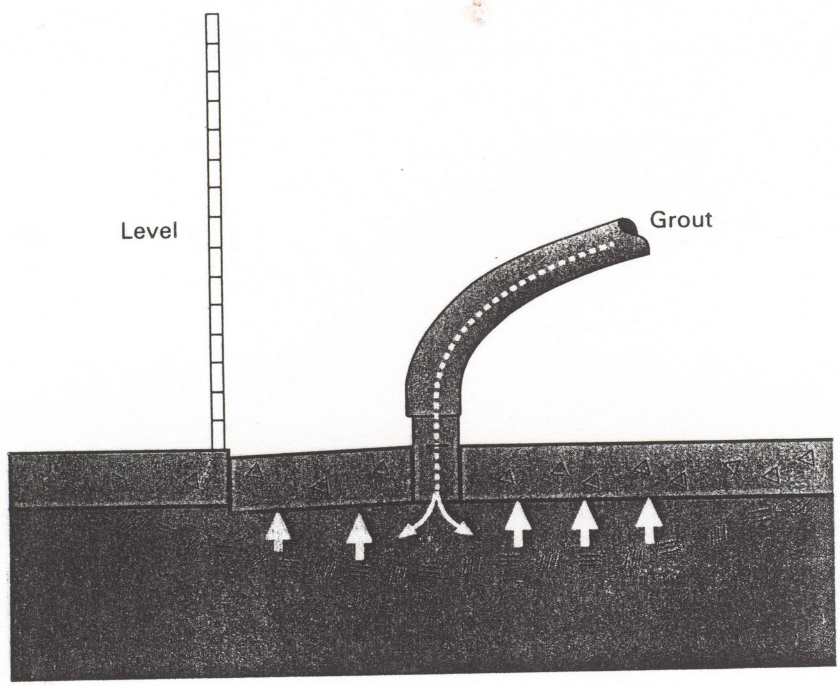


Figure 2. Slabjacking Schematic.

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 cement grouts 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, ultra-fine 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] It is essential to understand that the utilization of ultra-fine cement-based grouts – even if properly formulated and mixed – is alone not a guarantee of effective permeation in medium-fine sands. The other key properties of such a grout, namely stability (pressure filtration), and rheology (cohesion and viscosity), are equally important. These same principles apply to any cement-based grout and are the key to effective permeation.

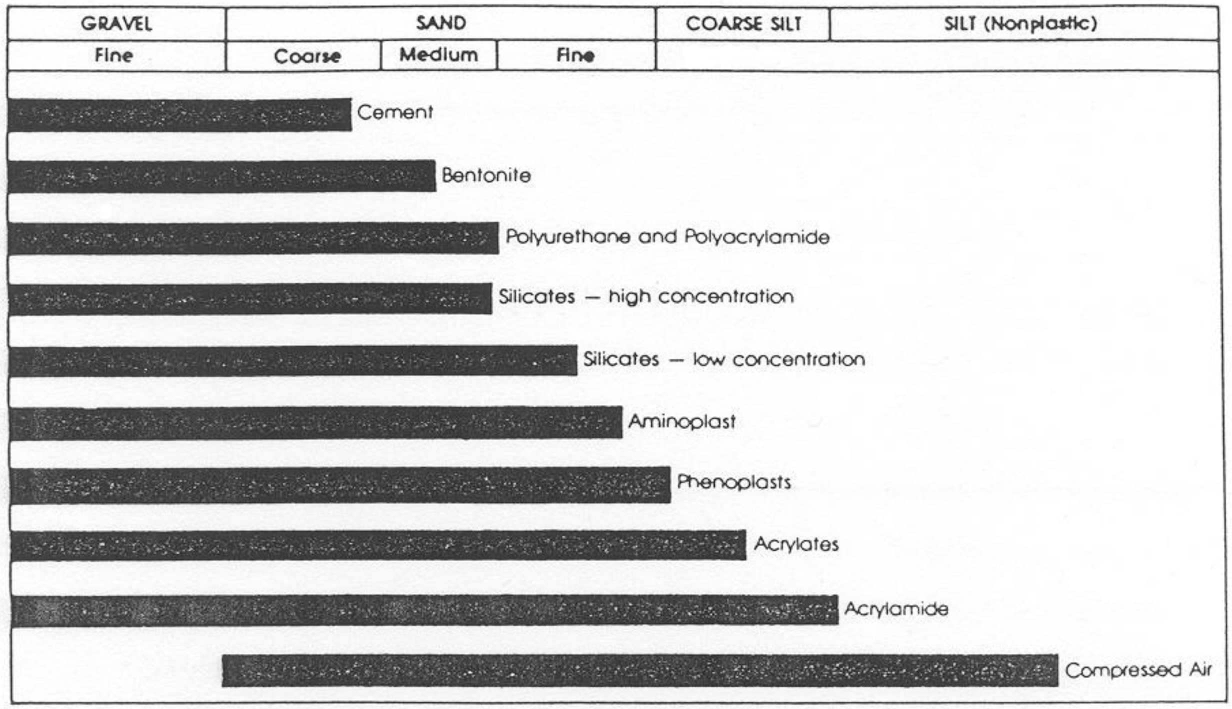


Figure 3. Penetrability of Various Grouts.[4]

Development of chemical grouting was a natural progression evolving from the limitations of early particulate grouting, such as large particle size, long setting times, instability, and poor 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 an 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] Such "two-step" processes are now obsolete.

In the 1950s, advances in polymer chemistry, aimed at reducing the two-step Joosten process to a reliable, single-shot system (i.e., two or more chemicals mixed prior to injection into the ground), 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, 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, while routine work continued with sodium silicate-based grouts. Responding to the concerns being voiced, the major domestic manufacturer of acrylamide grouts voluntarily withdrew the product from the market in 1978, though 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 was claimed even in 2003 that "virtually all construction grouting in the United States is done with silicates".(4) This situation is changing; sodium silicate is now being challenged by ultra-fine cement-based grouts due to concerns over permanency, practicality, and environmental aspects. However, in general, sodium silicate gels are still used in "borderline" conditions where ultrafines have not yet been demonstrably effective. 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; therefore, it is normal to attempt to fill larger pores first with conventional particulate grouts, and to permeate into finer or residual pores with chemical grouts, or ultra-fine grouts.

Recommended reading in permeation grouting includes:

- 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, 58]

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

Compaction Grouting

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 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 could 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] and as an integral component in the processes used to seal fast flows[60]. Compaction grouting features the use of low slump (usually 25 mm (1 in.) 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 surrounding soil. 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 that have been impacted. Compaction grouts can be designed as an economic and controllable medium for helping to fill large voids, even in the presence of flowing water.[50]

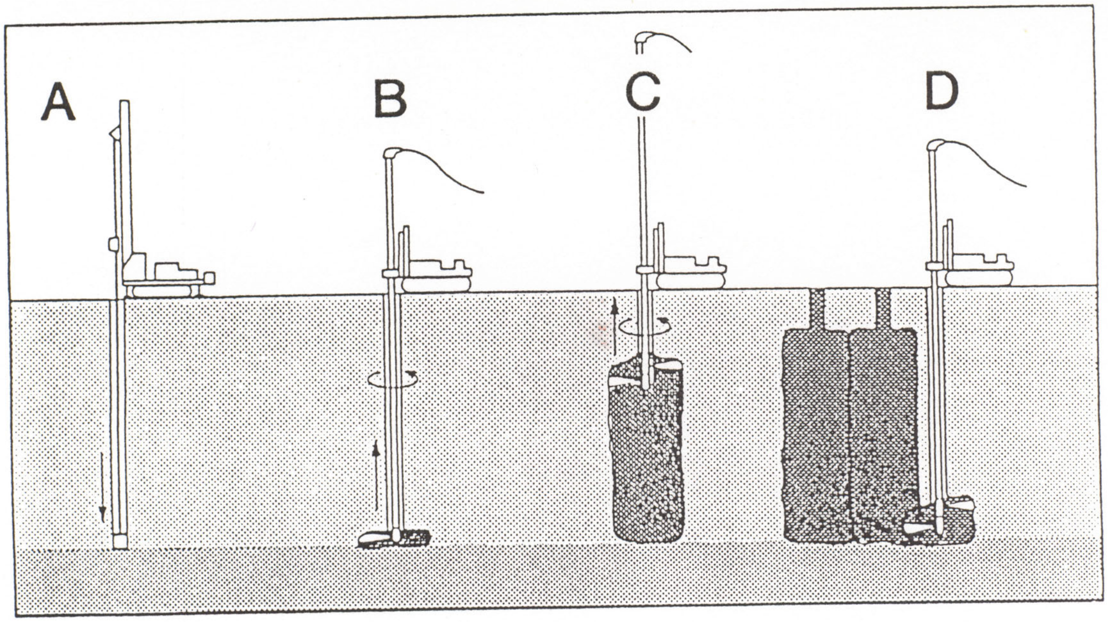
Recommended reading in compaction grouting includes

- "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]
- ASCE conferences 1982[9], 1992[18], 2003.[58]
- "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 by one specialty contractor 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 makes this a successful ground treatment technique, usable with almost any soil type. This is demonstrated in more than 200 successful projects completed between 1988 and 1997 in the United States. The rate of usage has increased substantially since then.

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 in Figure 5:



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]

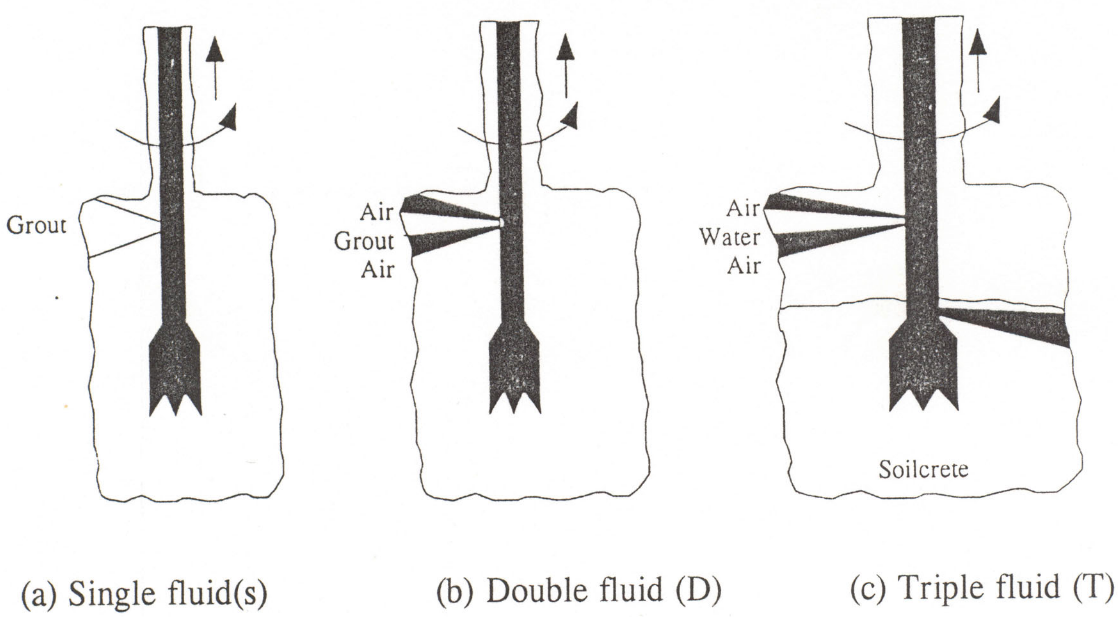


Figure 5. Different systems of jet grouting.

- One-Fluid System: The fluid is grout, and in this system the high-pressure (up to 50 MPa (7200 psi)) 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, although often lower strength.

- Three-Fluid System: An upper ejection of high-pressure water (30 – 50 MPa (4400 – 7200 psi)) inside compressed air envelope is used for excavation, with a lower jet (usually at lower pressure) emitting grout to replace the slurried 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.

Recommended reading in jet grouting includes:

- "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]
- ASCE Conferences 1982[9], 1992[18], 2003.[58]

Soil Fracture Grouting

In the course of 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 ("claquage"). 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 Sarnia, Ontario).[34] Most recently, the technology has been applied in a similar application for the new Metro in San Juan, Puerto Rico.

On the West Coast, less sophisticated “lense grouting,” as shown in 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]
- “The Development of Practice in Permeation and Compensation Grouting: A Historical Review (1802 – 2002) Part 2 Compensation Grouting.”[62]

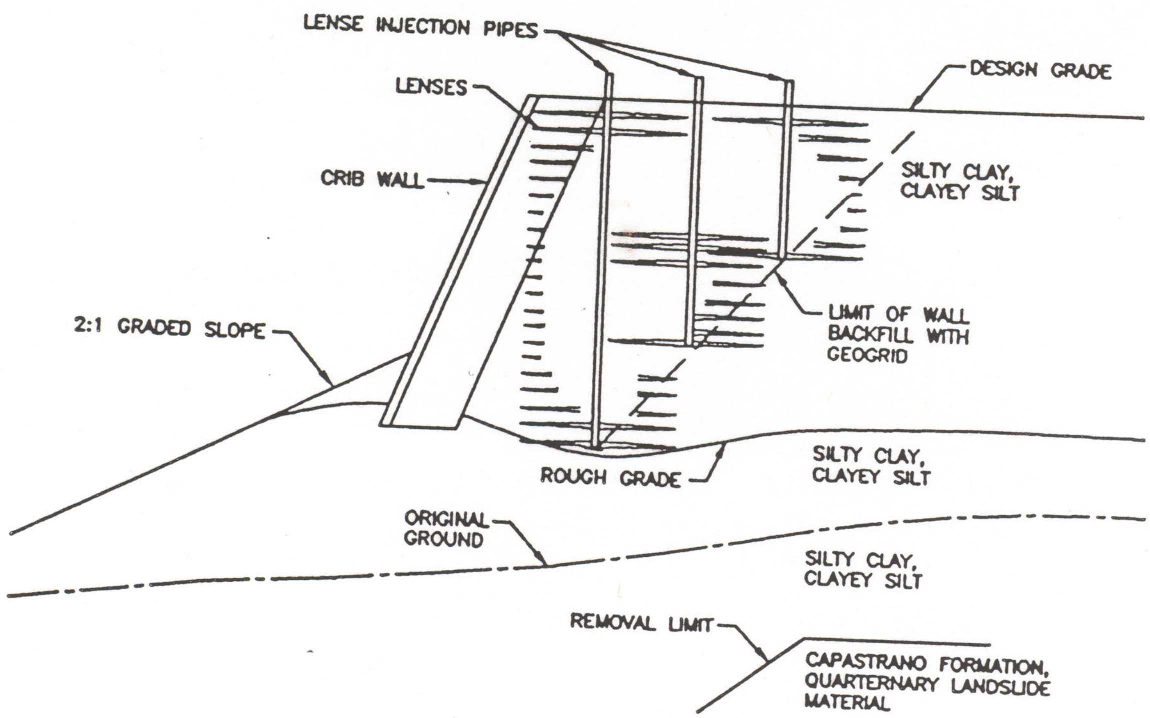


Figure 6. Lense Grouting Application.[33]

CHAPTER 2
DRILLING AND GROUTING PRINCIPLES, TECHNIQUES, AND MATERIALS

A wide range of grouting techniques is 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 that 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. Drilling and grouting data are best collected and analyzed in real time so that the response of the ground can be judged. The technique-specific elements include the equipment, hole spacing, construction sequencing, QA/QC, and monitoring. 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 direct connections between individual grout holes do not normally occur. In practice, primary grout hole spacings vary from 3 – 15 m (10 – 50 ft.), but there is no *universal*, constant distance. 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 acts 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 (1.5 in.) in diameter. However, for deep, inclined grout holes, larger diameter grout holes are frequently required, which require large diameter drill rods that are more rigid, resulting in a straighter bore. This is important at depths greater than 60 m (200 ft.), where deviation can otherwise be substantial. Grout holes over 150 mm (6 in.) in diameter are rare. The overall program design concept is, therefore, implemented in three phases:

- exploration, and site characterization
- construction with real time monitoring and data analysis
- verification of performance, via high order holes and other appropriate investigations

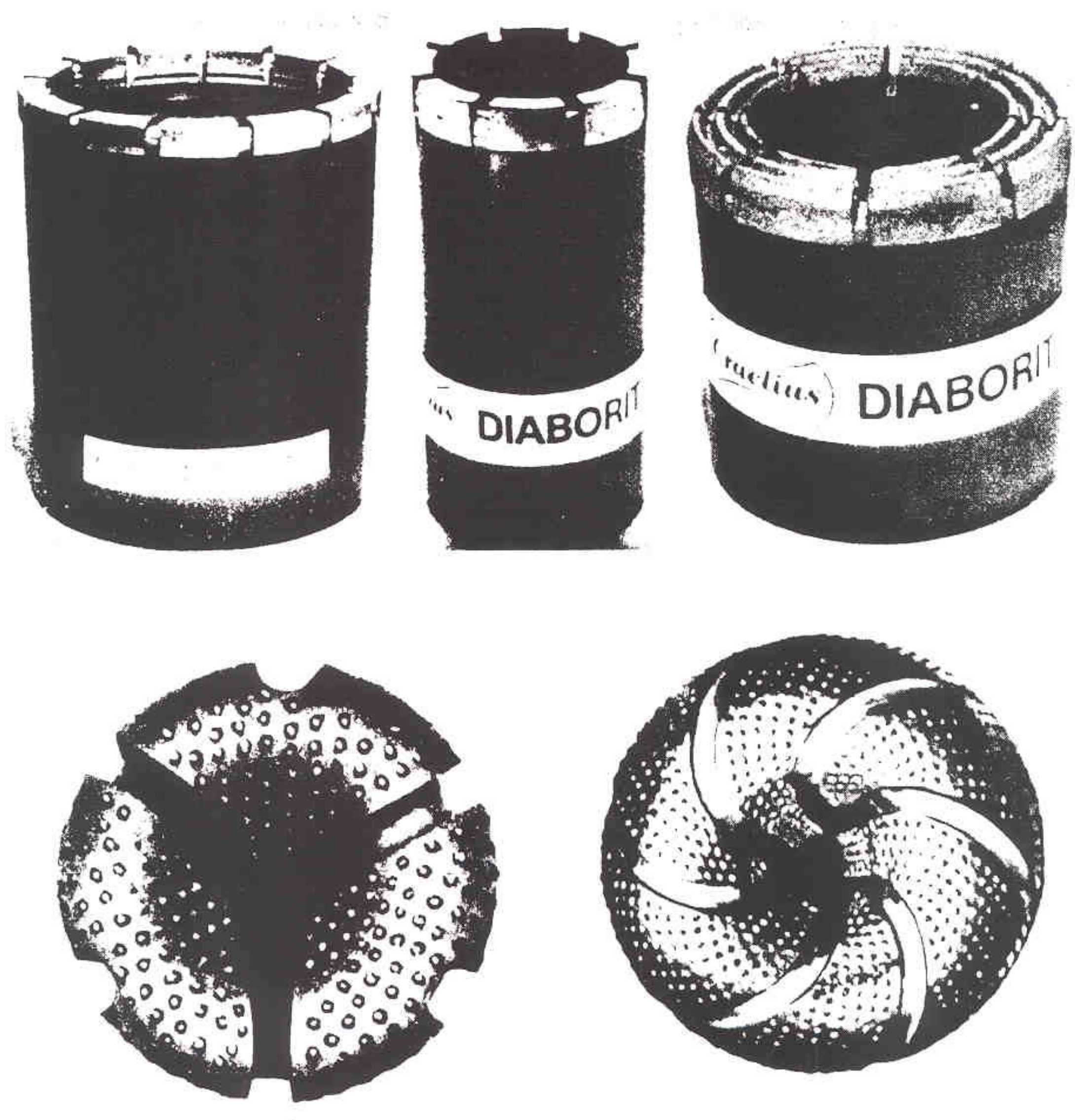
2.2 DRILLING METHODS, EQUIPMENT

These drilling methods are applicable for all three phases of the grouting program. A fuller description is provided in Bruce[63] and Weaver and Bruce[5].

Rock Drilling Methods

There are basically three methods of rock drilling:

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 (3 in.) diameter to depths of 50 – 150 m (160 – 800 ft.).



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 following:

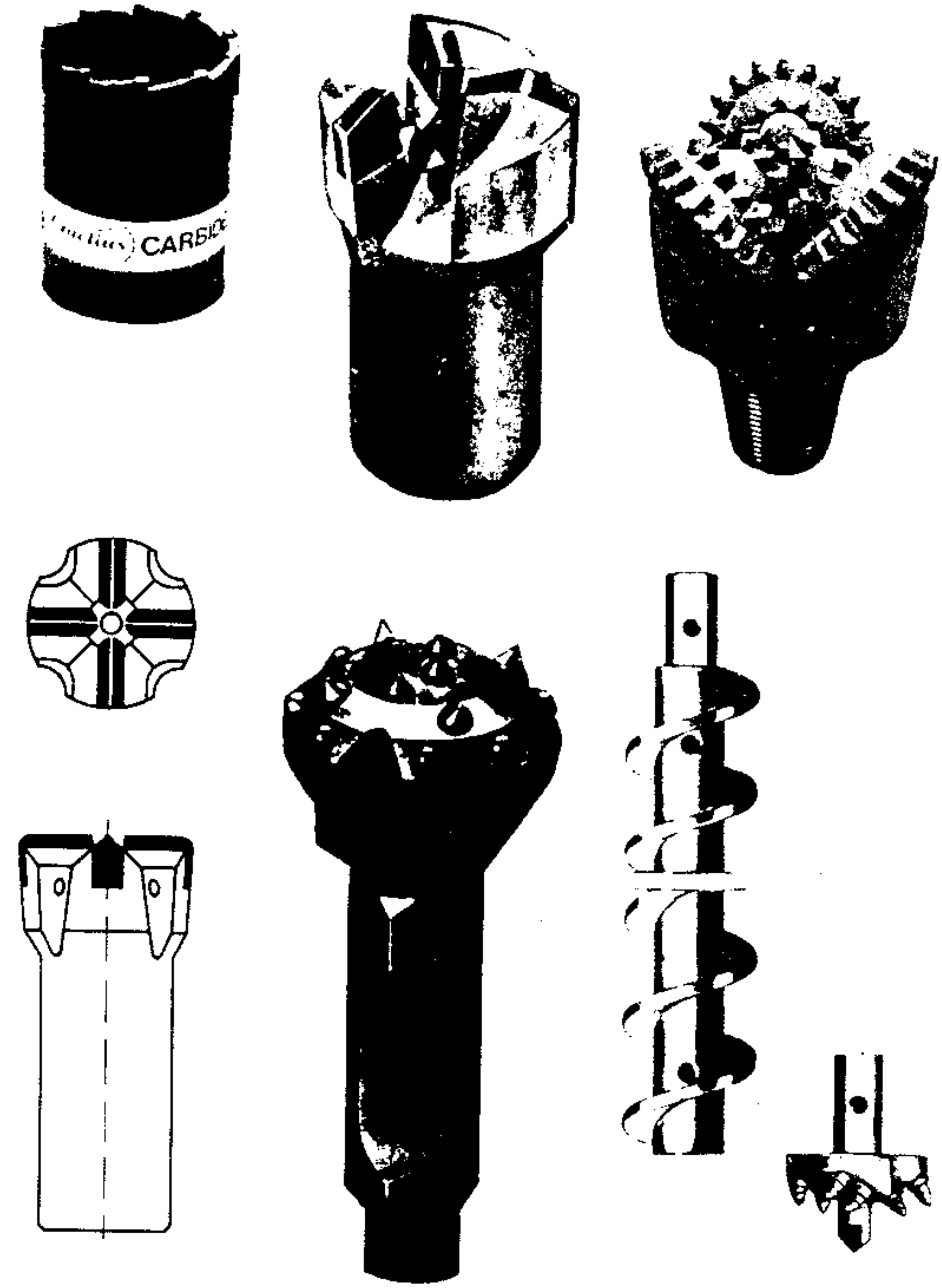
- The same equipment can be used for both investigatory and grout hole drilling.
- Continuous or intermittent exploration of the rock is possible over the entire length of the hole.
- Drilling can be done to relatively great depths.
- Straight holes are drilled with only little deviation.
- No or limited clogging of the rock fissures typically occurs. Cuttings are removed from the hole with the flush water.
- Since vibration is minimized, this is the preferred technique for drilling through existing masonry and brickwork.
- 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 that make subsequent packer installation easier.
- Good penetration speeds can be achieved in soft formations.

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 in Figure 8.

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 – 160 rpm in hole diameters seldom above 102 mm (4.0 in.). Hole depth is limited to approximately 60 m (200 ft.) by power, and by hole deviation concerns.

Down-the-hole drilling is performed at approximately 10 – 60 rpm in hole diameters of 85 mm (3.3 in.) and above to depths of over 100 m (330 ft.).



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. Thus, air-powered down-the-hole drilling is not acceptable for fissure grouting applications, although the speed and straightness benefits of the principle can still be exploited by the new generation of water powered hammers.

Advantages of percussion drilled grout holes include the following:

- Higher and consistent penetration rates can be maintained in rock, as compared to other methods.
- Smaller and lighter drill rigs can be used; these are easily moved from hole to hole on the surface.
- Low drilling costs can be achieved, as 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 with relatively constant penetration rates.

In principle, the prime controls over the choice of drilling method should ideally be related to the geology, the hole depth, and the 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, and water powered, down-the-hole hammers offer significant cost and technical advantages.

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.

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.

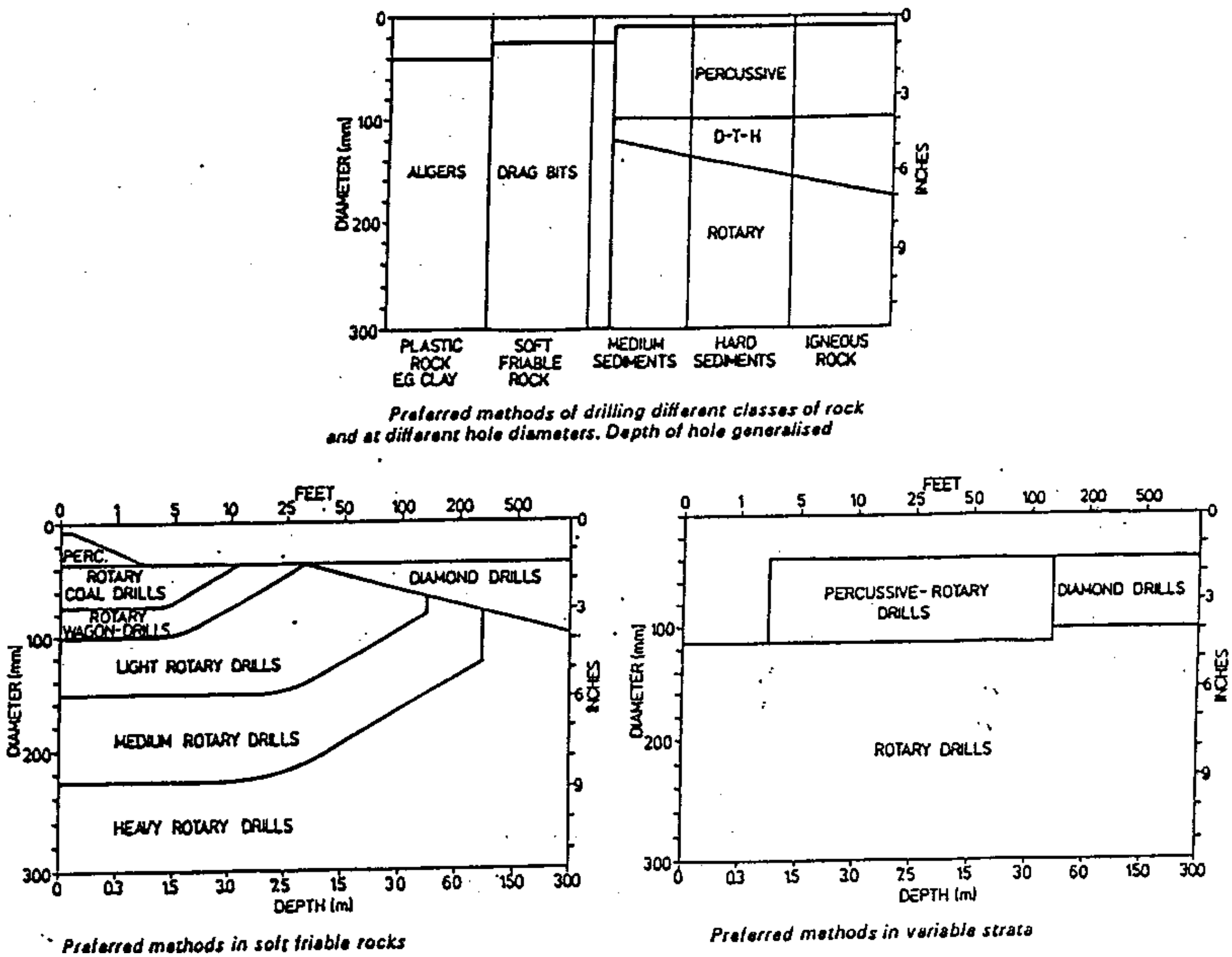


Figure 9. General Guides for Selecting Drilling Method and Equipment for Rock Drilling.(53)

Larger diameter 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 as follows:

- High speed rotary drilling: normally 2 – 5% to depths of 80 m (260 ft.).
- Top hammer drilling: long holes – 15 – 20% (with guide rods, under 5% can be reached); shallow holes, down to 12 – 15 m (40 – 50 ft.) – 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 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 toward 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, and is used to provide a broad categorization of the ground. 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.

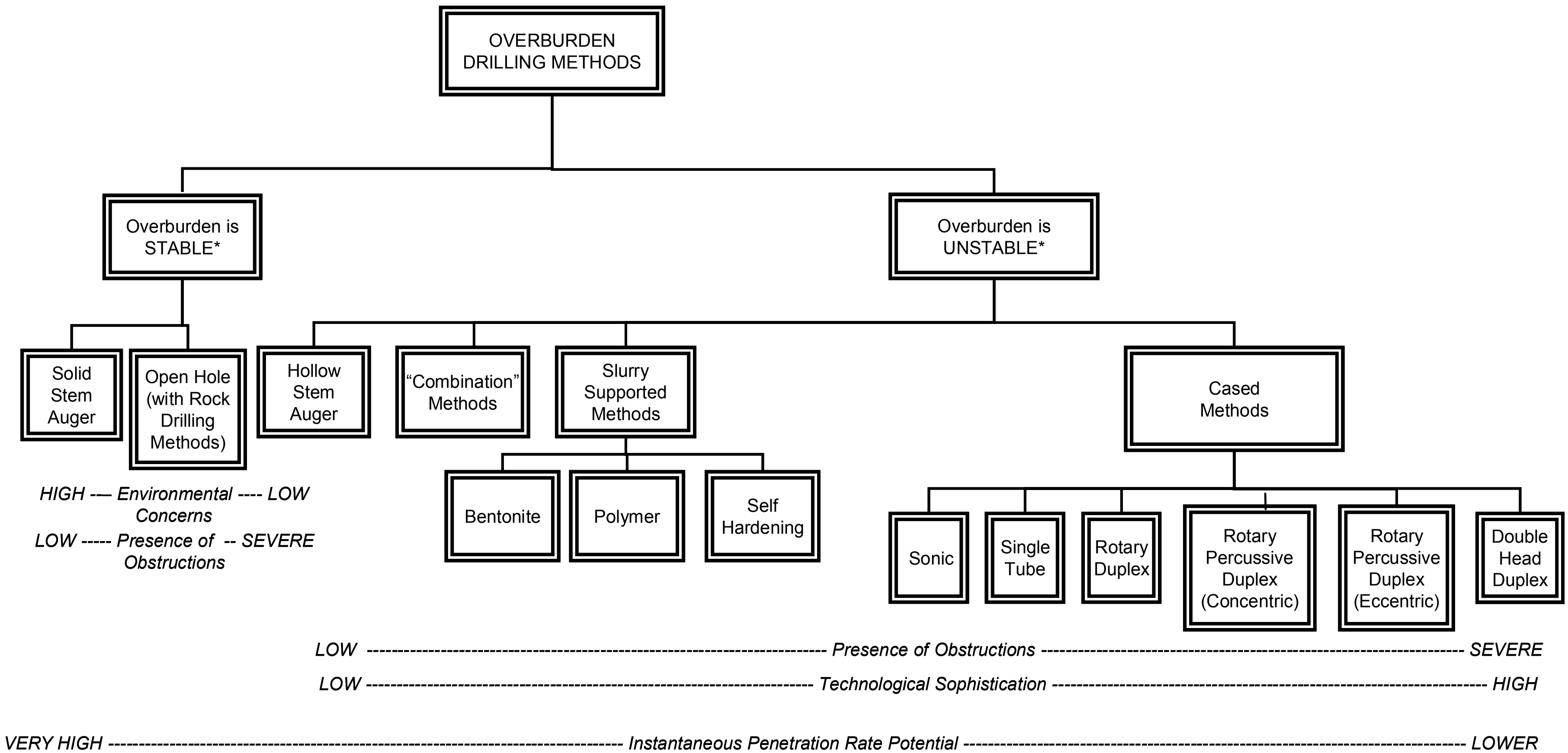
For example, the dividend of the flush water flow and the flush water pressure can be used to locate major fissures and cracks: when the drill bit hits a fissure, the pressure will drop and, at the same time, the flow will increase. This is due to the inflow of the flush water into the fissure. The data may be electronically generated 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.

Soil Drilling Methods

There is a wide range of overburden drilling systems, as illustrated in Figure 10.

The choice of method should satisfy the geometric requirements of the drilling, and be consistent with the geotechnical and environmental challenges of the soil.

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. The choice of flush is critical, especially cohesionless materials below the water table: air should never be used in such circumstances. Most recently, sonic drilling has become very popular, especially for applications demanding absolutely minimal damage to the surrounding soil (e.g., penetrating through an existing embankment dam). This method is fast, reliable, and uses no, or very little, flush. Further details are provided in Bruce[63].



*Stability refers to the overburden's ability to maintain the shape and size of the drilled hole without detriment to the surrounding ground after withdrawal of the drilling system.

Figure 10. Classification of Overburden Drilling Techniques.

2.3 GROUTING MATERIALS

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

1. Particulate (suspension or cementitious) grouts, having a Binghamian performance. (See Figure 11)
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, fly ash, 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 when excess pressure is applied.

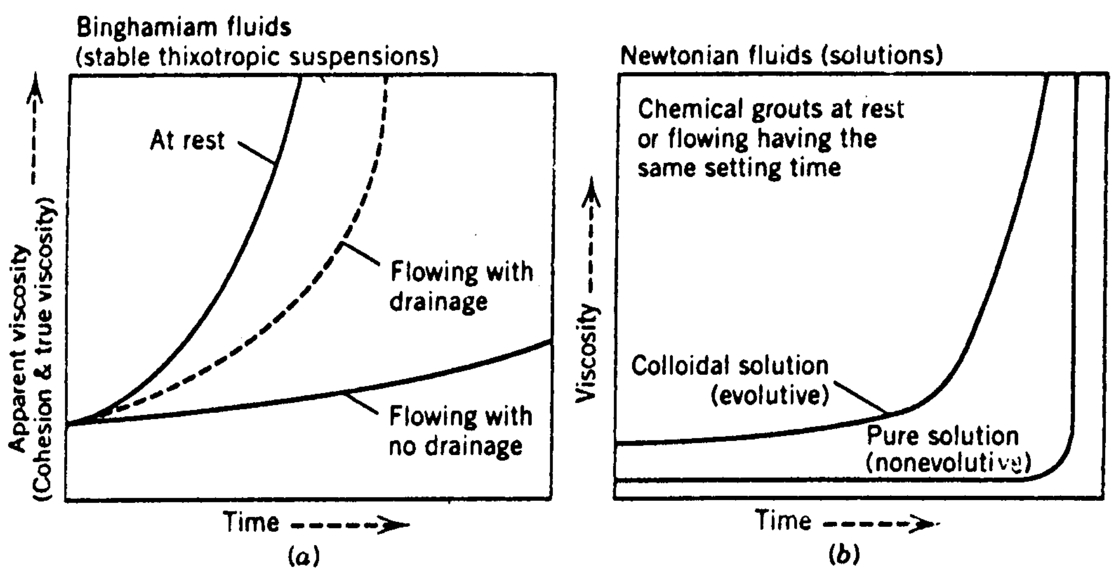


Figure 11. Rheological Characteristics of Major Families of Grouts
a) Category 1, b) Categories 2 and 3.[65]

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 that 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[(59)].

A summary of some characteristics and material costs is provided in Table 1.

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
- 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.

Table 1. 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 fissures
Type III Cement	Med (15 cps-2:1)	Low	High	>$0.05	Non-flexible; penetrates only slightly smaller fissures
Ultra-fine Cement	Low (8cps-2:1)	Low	High	$0.30	Non-flexible; penetrates fine fissures
COLLOIDAL SOLUTION:					
Silicates	Low (>6cps)	Low	Med	>$0.13	Penetrates fine fissures
SOLUTION GROUTS:					
Lignosulfites	Med (>8cps)	High	Low	>$0.26	Flexible; penetrates fine fissures
Polyrethane	High (>400 cps)	High	High	>$1.32	Flexible; penetrates large fissures
Acrylamides	Low (1.2cps)	High	Low	>$0.53	Flexible; penetrates very fine fissures
Acrylates	Low (1.5cps)	Low	Low	>$0.53	Flexible; penetrates very fine fissures

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 than 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 microns, with the bulk being less than 4 microns. Examples of the gradation curves from some of the many types now available in the U.S. are shown in Figure 12.

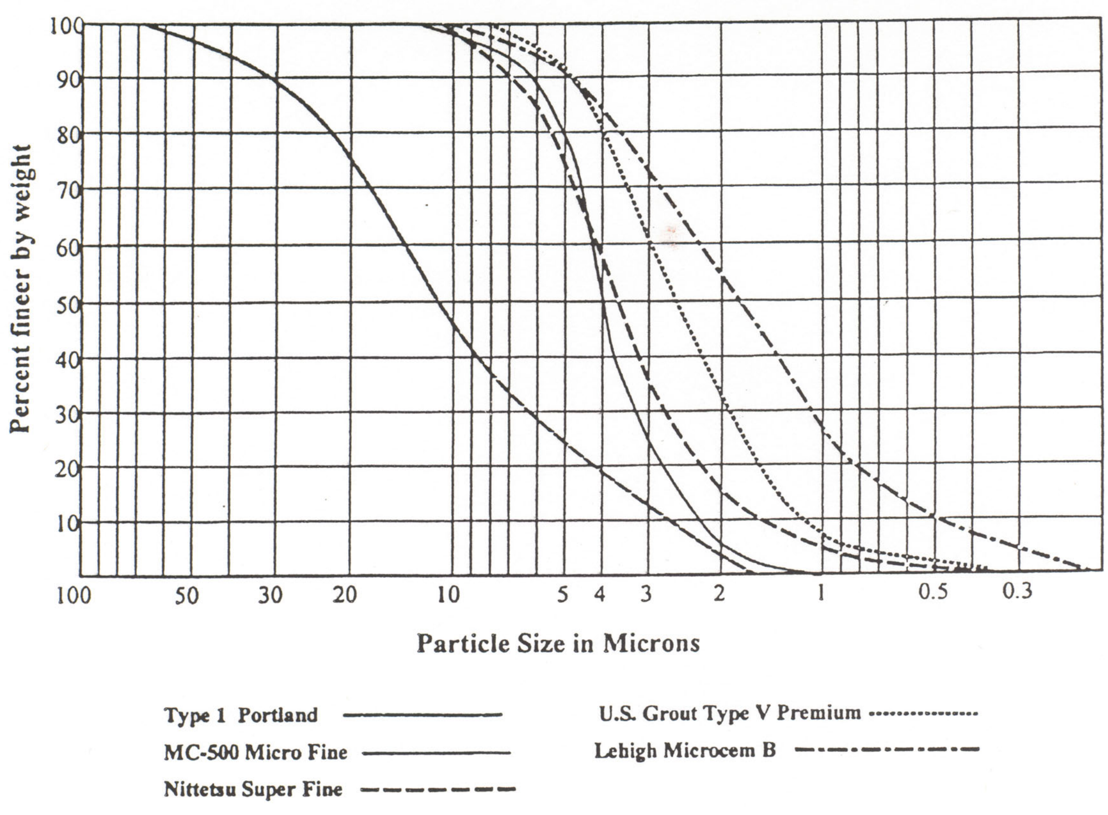


Figure 12. Grain Size Distribution for Various Cements.(11, 64)

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 (ºBé). Typically R_p is in the range 3 – 4, while specific density varies from 30 – 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 – 60 minutes), low strength (less than 2 MPa (290 psi)), and poor durability. However, they may prove locally acceptable for temporary applications, say less than a few months. Sodium silicate solution without reagent may be used to accelerate the stiffening of cementitious grouts, a traditional defense against fast flows in small orifices.

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 2. Chrome lignosulfonates are not discussed, because of the environmental damage caused by the highly toxic and dermatitic components.

Table 2. 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 (220 psi) 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 – 100 cP. They may be either
  - ***Hydrophobic:*** react with water, but repel it after the final (cured) product has been formed, or
  - ***Hydrophilic:*** 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 – 1,000 cP and strengths as high as 2 MPa (300 psi) . 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 – 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(60).

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

- far lower permeability;
- far superior creep behavior of treated sands for grouts of similar strength (2 MPa (290 psi));
- a permanent durable filling is assured, even if an unusually large pore space is encountered, or a large hydrofracture fissure is created.

2.4 GROUTING EQUIPMENT

Many types of grouting equipment are commercially available and are used routinely for grouting operations of different types and scale. Each major rock and soil grouting technique basically demands its own specialized equipment. However, 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 an HMG (high mobility grout) injection application is shown in Figure 13.

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 far superior to standard slow-speed mechanical mixers because they produce grouts of greater uniformity and quality more quickly. 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. The use of the metric system is numerically advantageous in grouting calculations.

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 HMG, concrete pumps for compaction grouting (modified as necessary), and custom built equipment for chemical and jet grouting.

Technique-specific aspects regarding equipment are addressed in Chapters 3 – 5. Typical examples of drilling and grouting equipment are shown in Figures 14 – 26.

2.5 QA/QC PRINCIPLES AND VERIFICATION

It is often overlooked that every production 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 that are taken and any tests (e.g., permeability) that 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 routine and responsive fashion.

Similarly, the grouting data provide 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 routinely 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.

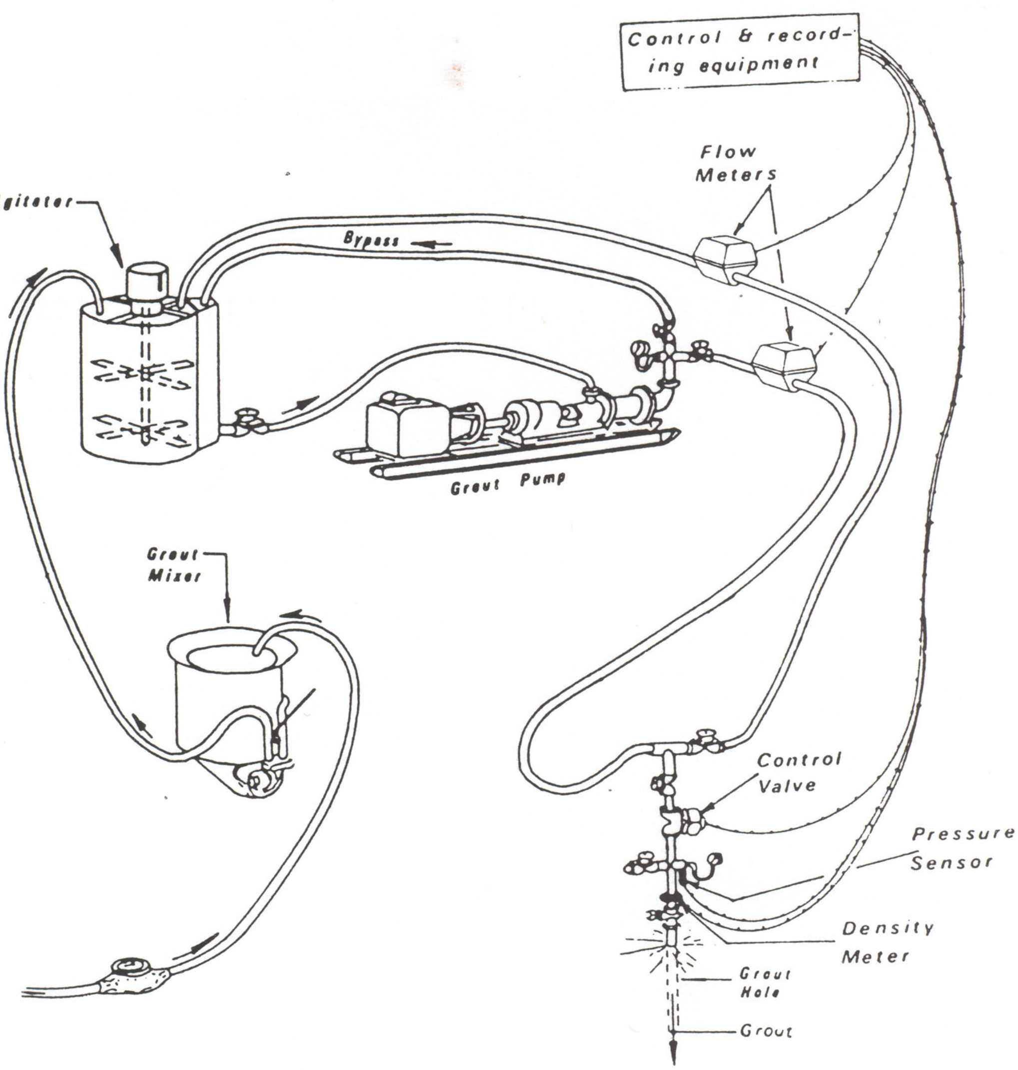


Figure 13. Typical Layout for HMG Injection.(3)

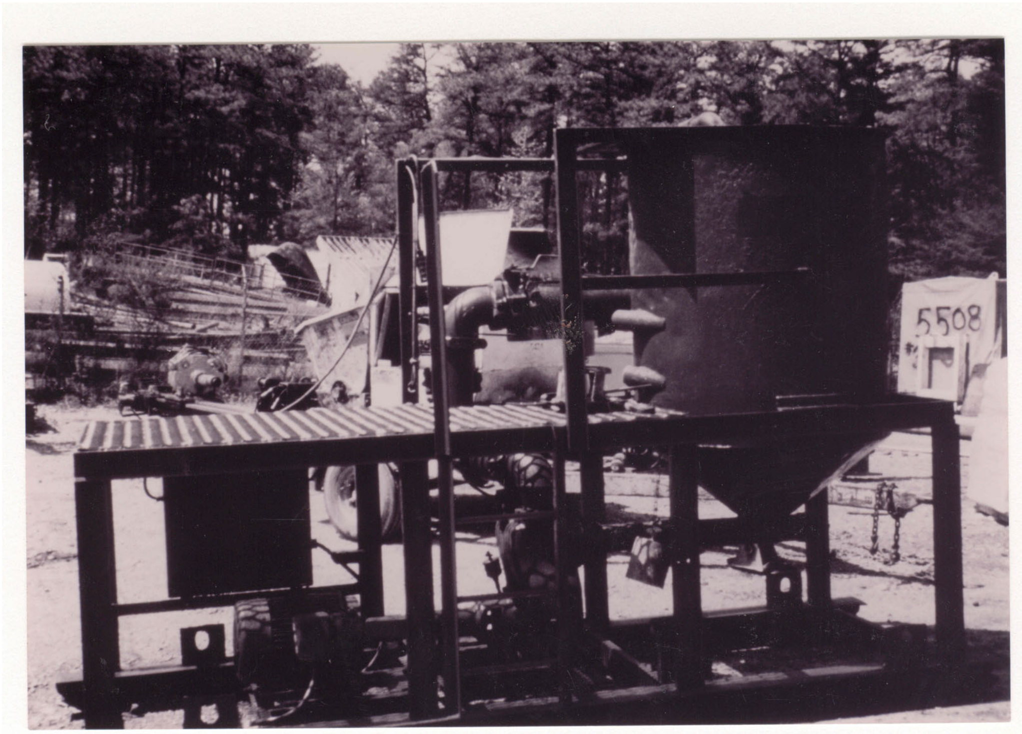


Figure 14. Electric-Powered, High-Shear HMG Grout Mixer.

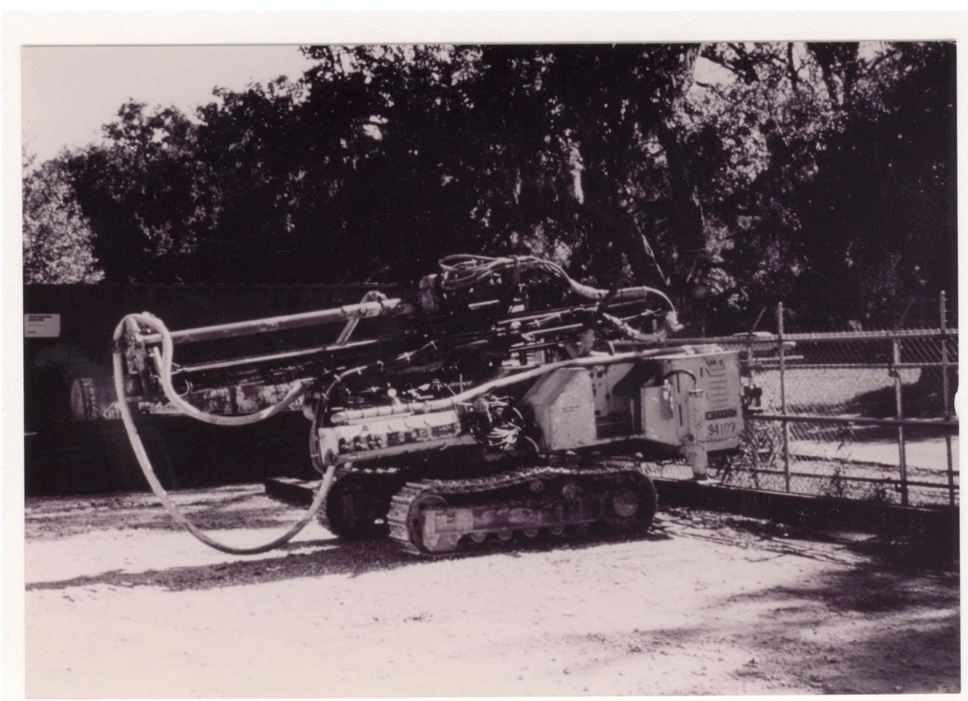


Figure 15. Rotary/Rotary Percussion Diesel Hydraulic Track Drill.

Figure 16. Small Compaction Grout Batcher.

Figure 17. Larger On-Site Grout Batching Plant for Compaction Grouting.

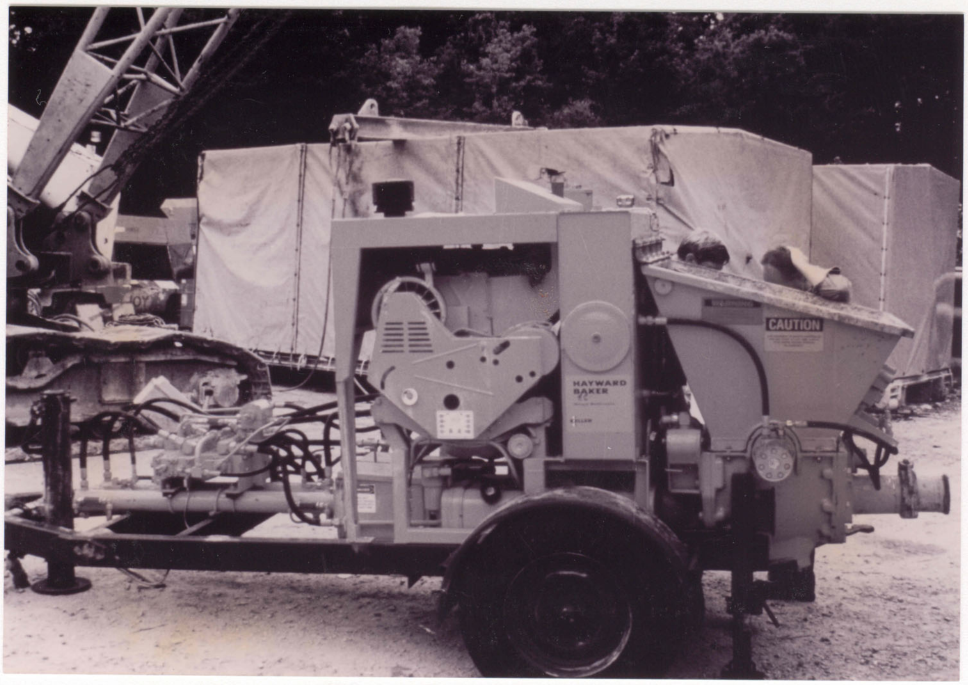


Figure 18. Compaction Grout Pump.

Figure 19. Compartmentalized Tanker for Raw Chemical Grout Components.

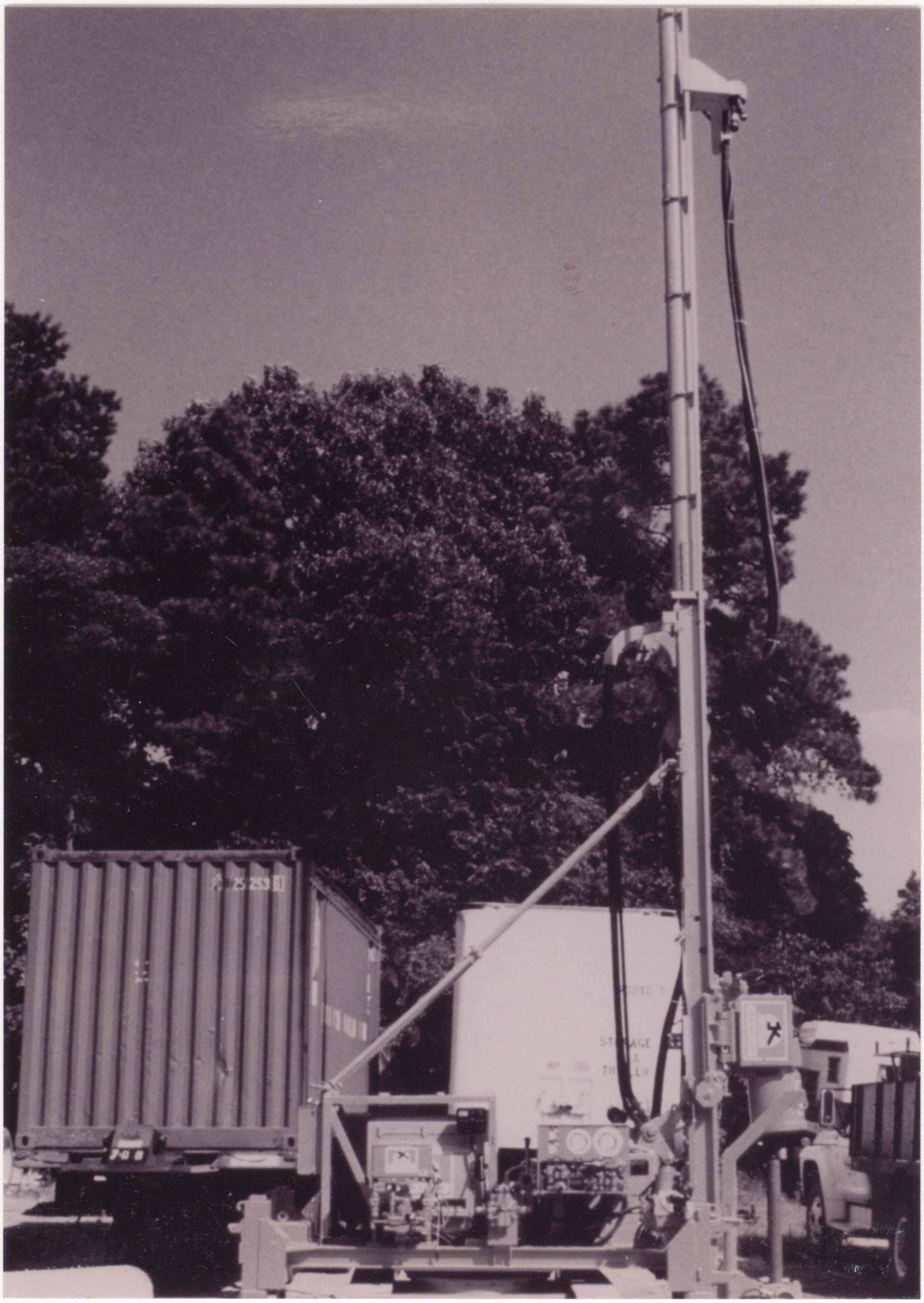


Figure 20. Jet Grouting Rig.

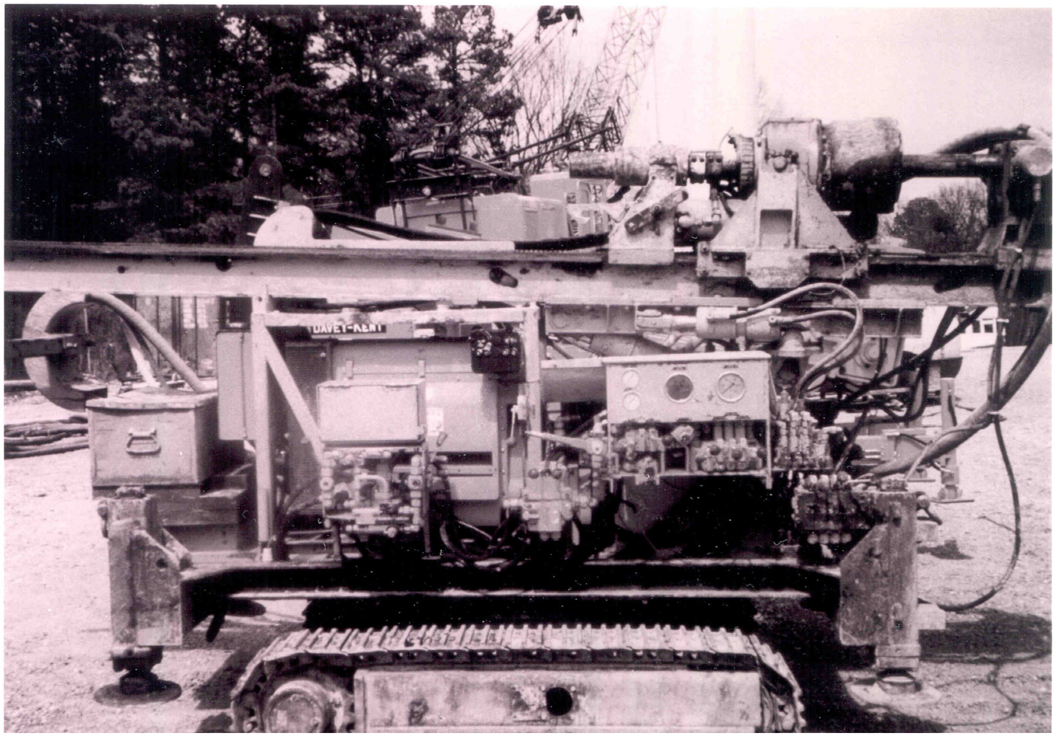


Figure 21. Jet Grouting Rig (drill mast at rest position).

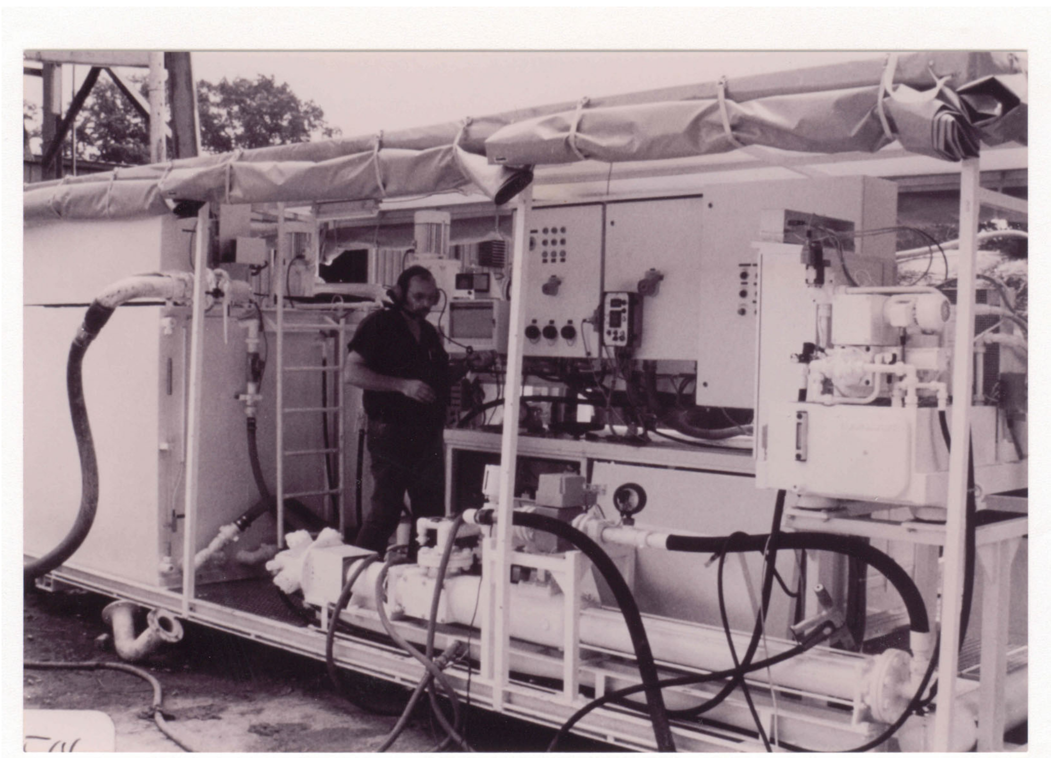


Figure 22. Pumping Station for Triple Fluid Jet Grouting.

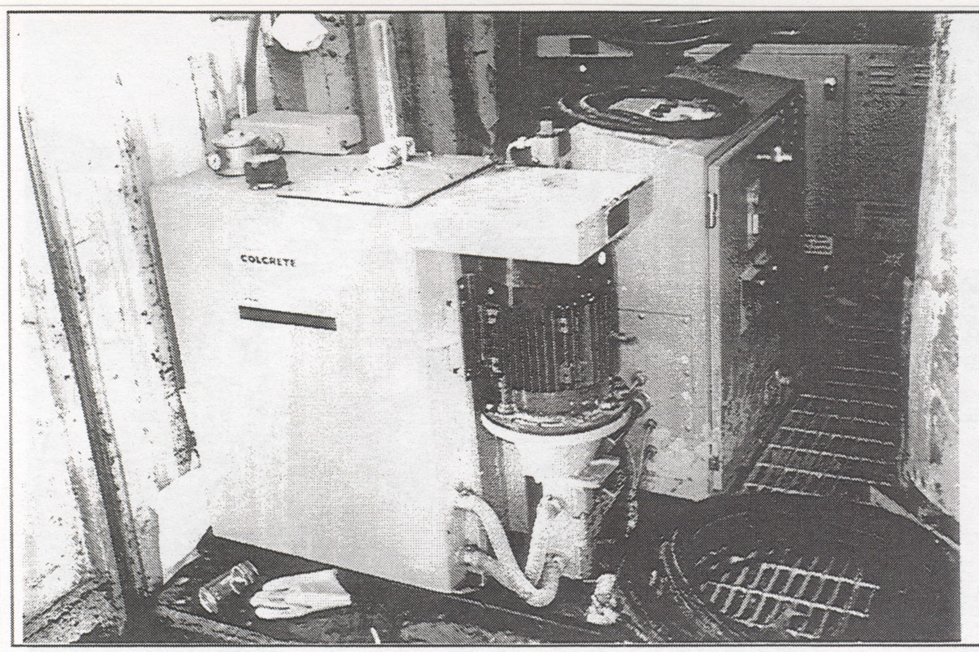


Figure 23. High Pressure HMG Grout Pumping Unit.

Figure 24. Hydraulic Crawler Rig for Lime Injection.

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, jet grouting, and permeation grouting, are well established. Others, like 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, waterproofing is construed to be used in conjunction with new construction, and water control to be used in conjunction with remedial applications. Applicable techniques are permeation and jet grouting, while compaction grout can also be used for both water control and waterproofing.

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 HMG grouting has successfully raised slabs and footings, its major disadvantage is the lack of control of the fluid mixes. Both compaction grouting and soil fracture grouting can be precisely controlled for structural settlement remediation or compensation.

Settlement Control

Depending on the soil type, cost and potential cause of settlement, permeation, compaction, jet and fracture 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 remediated by permeation, jet grouting, and the use of low mobility 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 immediately 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 25.

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 non-specialist engineering community.

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 that bypass the problem soil, a soil grouting solution uses the existing soil, improving it by grouting to make up for the soil's deficiencies.

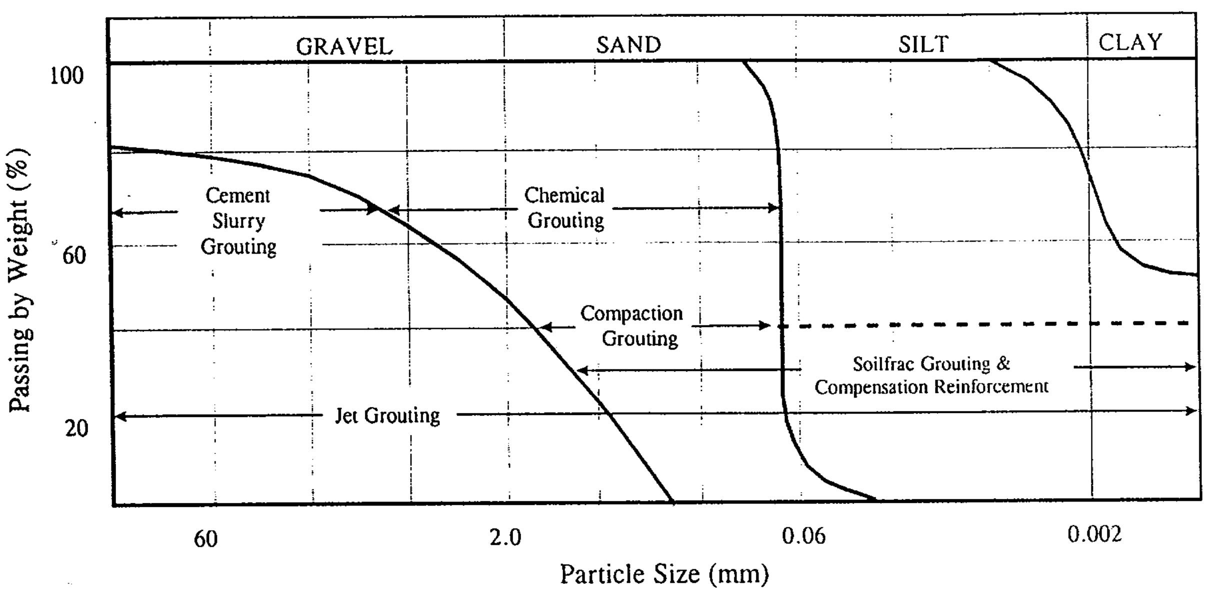


Figure 25. Range of Applicability of Soil Grouting Techniques.

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 26 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, density, 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 regime.

Stratigraphy

Stratigraphy, including the variations in soil properties, especially permeability of 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 (14 in.) drive samples should be obtained for every 750 mm (30 in.) of hole, instead of one sample per 1.5 meters (5 ft.), 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.

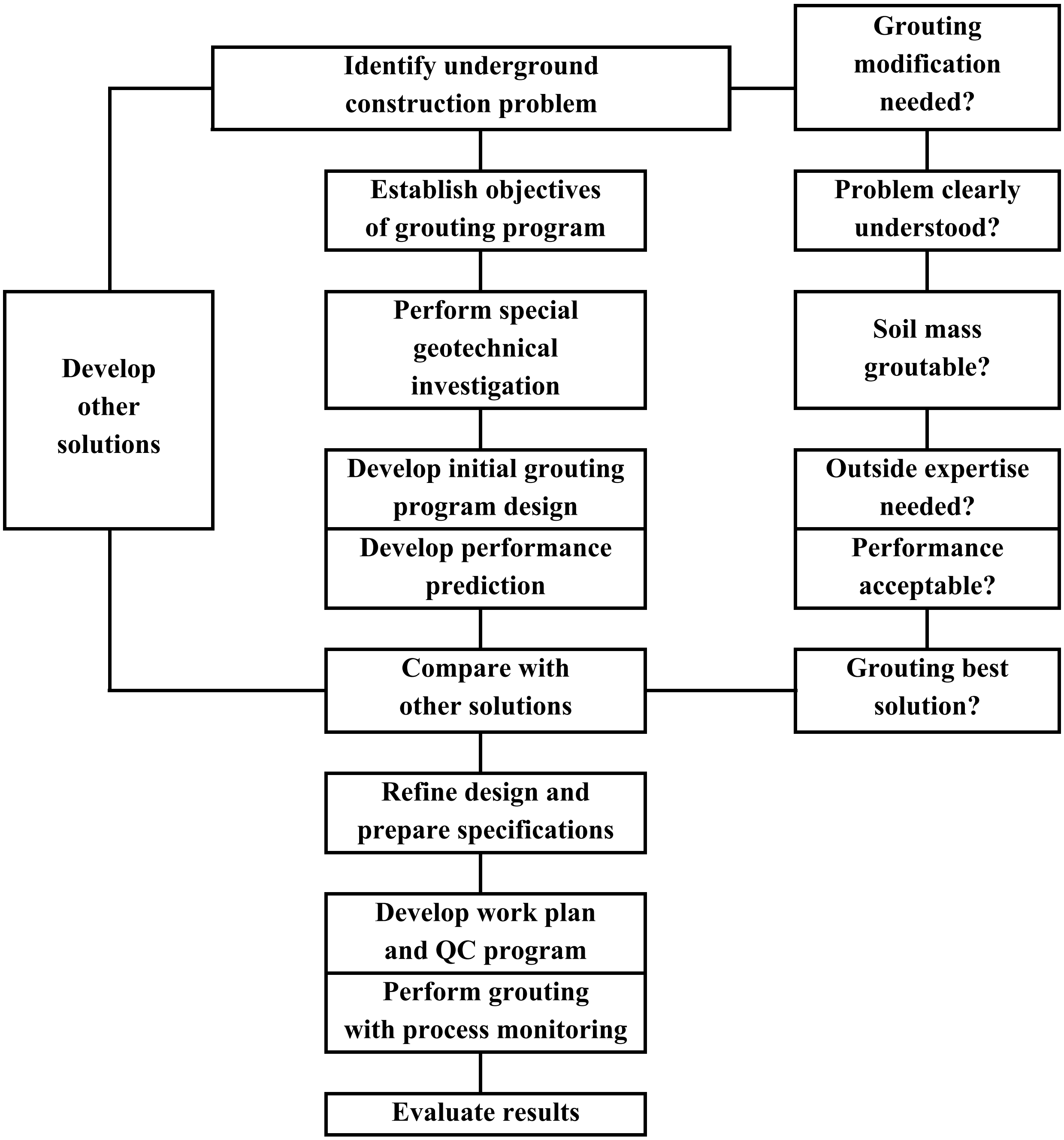


Figure 26. Grouting Decision Flow Chart.

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.

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 pressures that will be created within the grouting zone. Active drain lines and sewers should be monitored to detect any grout infiltration, e.g., by using recording pH meters fitted with audible alarms that are activated when effluent pH reaches a certain level.

Groundwater

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 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 tunneling to increase cohesion, as shown in Figure 27
- liquefaction retrofit mitigation by increasing density and displacing pore water

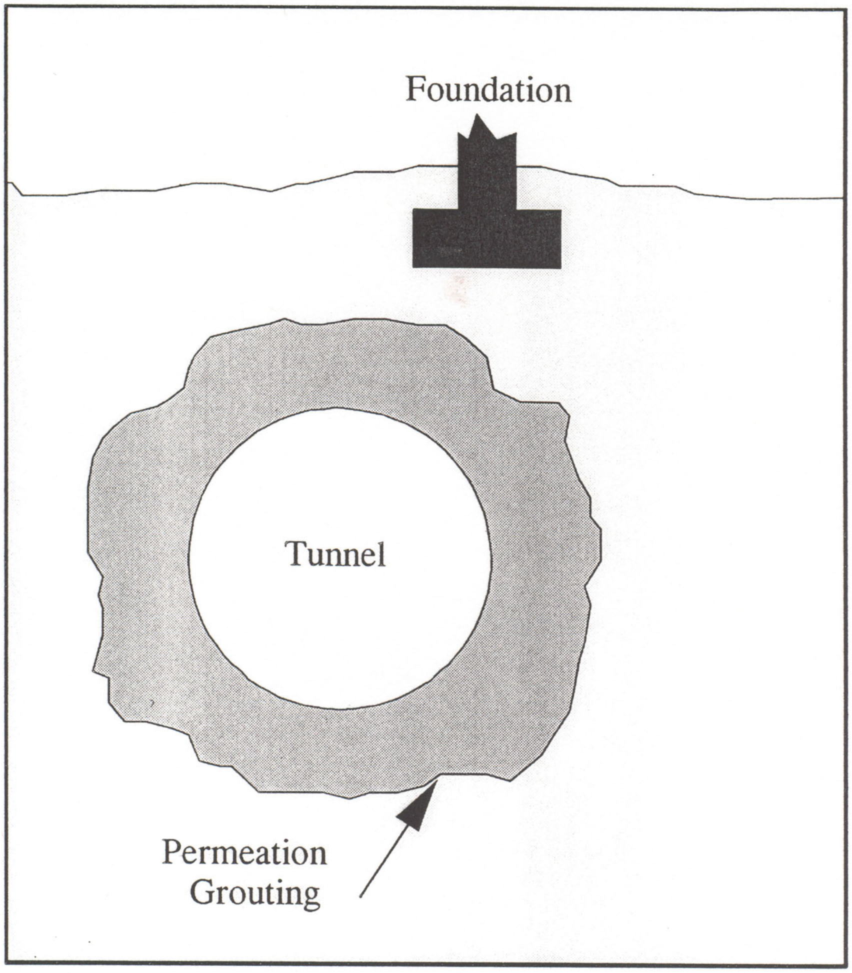


Figure 27. Tunnel Excavation Support Using Permeation Grouting.

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% 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% silts and clays.*

Practical injection rates range from about 2 – 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 micron sieve (# 200). *Soils are initially classified as readily groutable if they have less than 12% fines, moderately groutable for 12 – 15% fines, and only marginally groutable for 15 – 20% fines.* Sands are usually considered ungroutable if they have more than about 20% fines. Figure 28 shows typical grain-size ranges for soils amenable to permeation by typical silicate grouts.

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 74 micron (#200) 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]

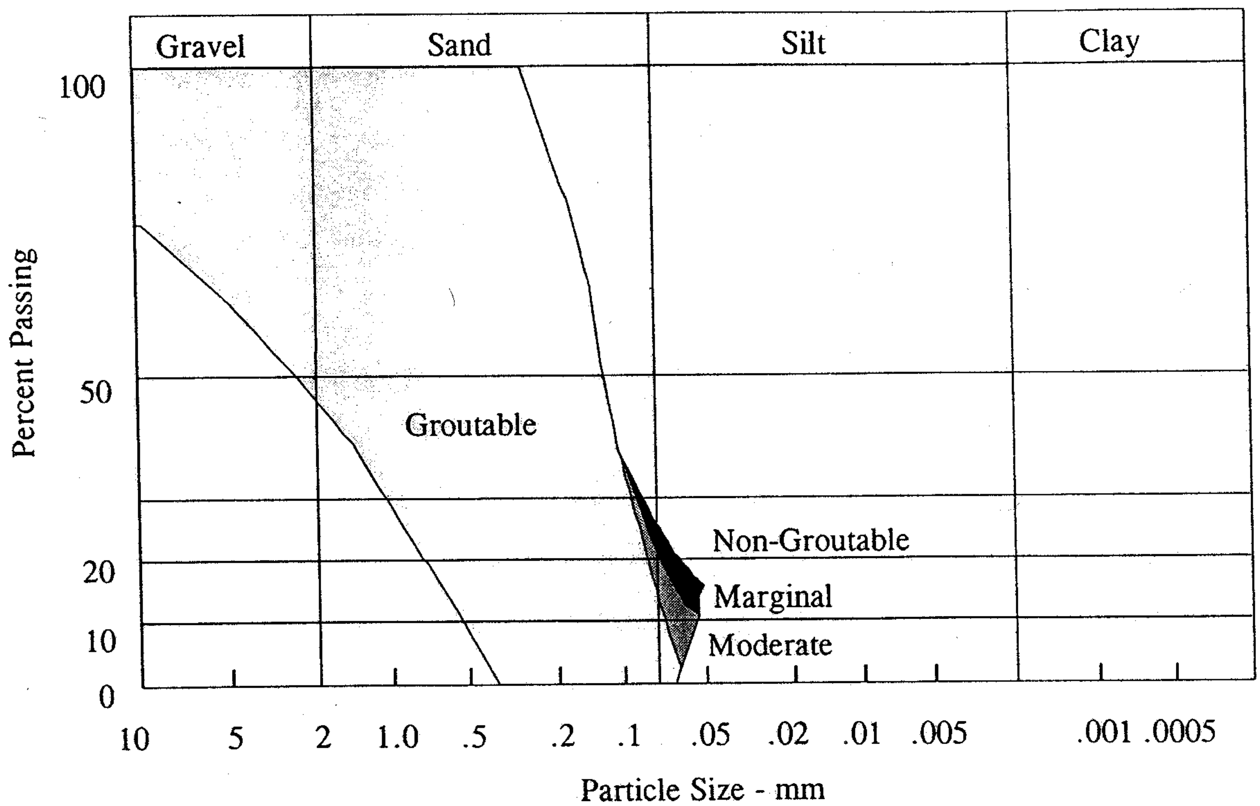


Figure 28. Typical Grain-Size Curves for Soils That can be Permeated.

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.

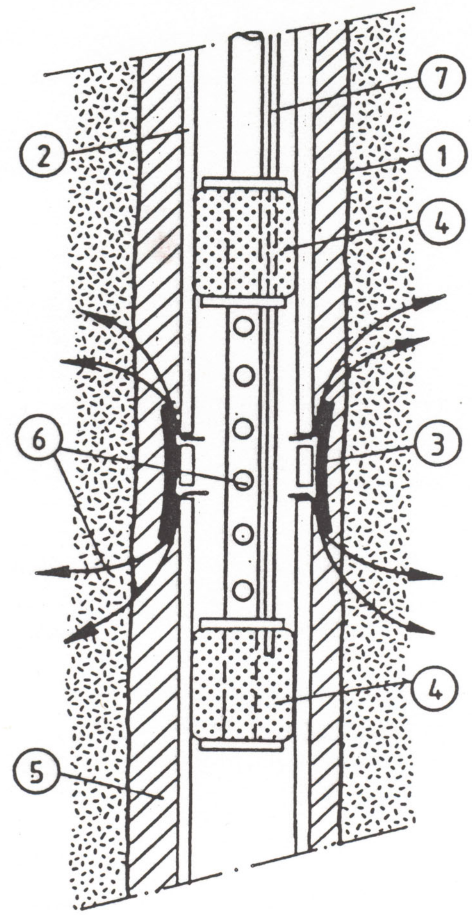
The early establishment of clear, quantitative objectives to be achieved by a permeation grouting program is a basic prerequisite to good design and satisfactory, economical performance.

Program Design

These are well-defined equations that 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 – 1.5 m (1.6 – 5 ft.) for waterproofing and from 1.0 m – 1.6 m (3.3 – 5.2 ft.) for structural applications. A typical figure for each is 1.2 m (4 ft.).

- **Equipment:** Sleeve-port grout pipes, also called "tubes-à-manchette" as shown in Figure 29, 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, usually 33-cm (1 ft.) centers. The grout holes are covered with a rubber sleeve that acts as a one-way check valve. The grout pipe is installed in a slightly oversized 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 by grouting pressure 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 may be 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.



1 Wall of borehole
2 Tube-a-manchette
3 Opened valve (manchette)
4 Double packer

Figure 29. Mode of Operation of Tube-À-Manchette.[7]

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.

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 (finer grained) to 0.45 (coarser grained), 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 grout. Depending on the scale of the project, it will be prudent to add an extra 5 – 15% grout volume to compensate for "edge dilution" effects.

- **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 – 50 minutes. Batch-type systems can also be used with appropriate QA/QC procedures. 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. Batch mixing does not require such “Y” fittings, as the reacting grout is pumped to the hole through one line.

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% grout volume factor. Convert to liters by multiplying by 1,000 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 ($2.50/gal) in-place can be used for estimating purposes.

A mobilization/demobilization rate ranging from $10,000 – $50,000 and a cost of providing and installing the sleeve port grout pipes starting at $65 per linear meter ($20/lft) 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 (740 ft.) 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 – 5 m (6.6 – 16 ft.) 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 (43 ft.). Because of the potentially flowable nature of the clean, granular soils, the narrow 1.8 m (6 ft.) pillar between the 6.4 m (21 ft.) diameter tunnels (Figure 30) and the shallow ground cover of 5.8 m (19 ft.) at the freeway and 3.7 m (12 ft.) 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 31).

Along the freeway where traffic disruption was deemed unacceptable, horizontal drilling and grouting was required, with sleeve-port grout-pipe lengths ranging from 79 – 97 m (260 – 320 ft.). 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 (40 ft.) 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 (0.4 in.), 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 (740 ft.) 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 32).

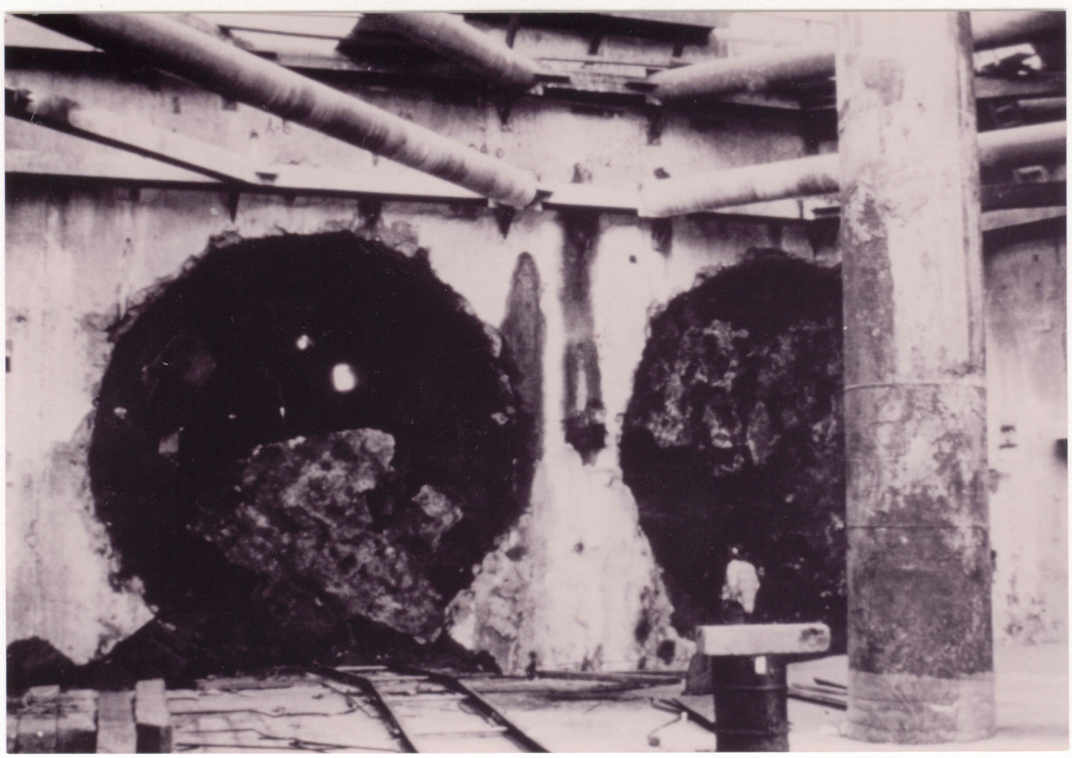


Figure 30. Los Angeles Subway Twin Tunnel

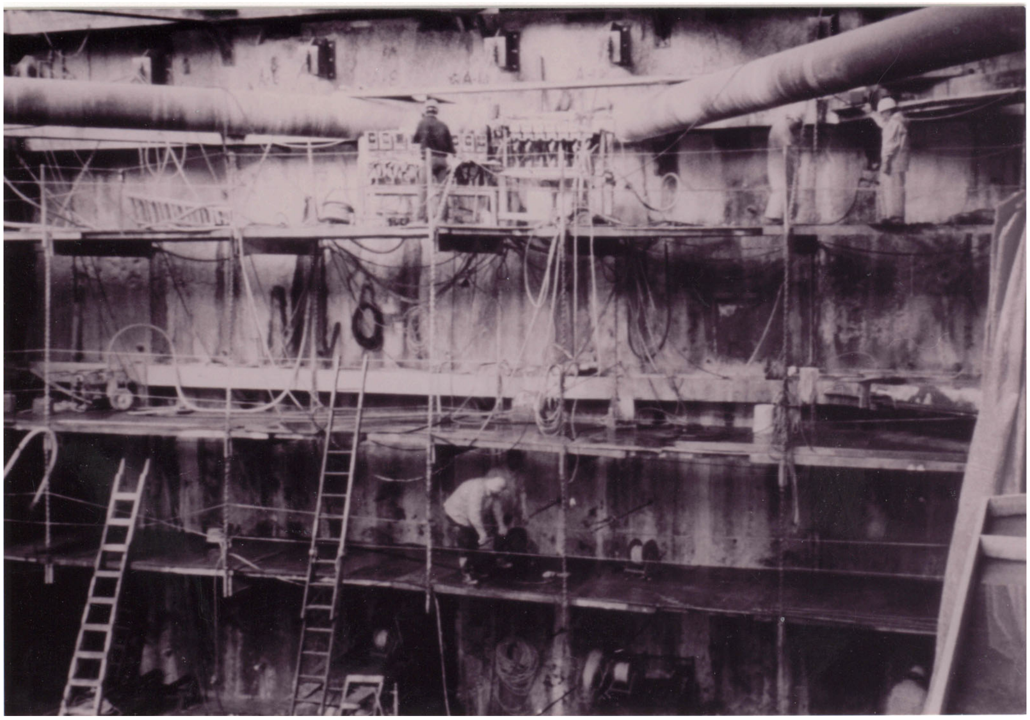


Figure 31. Full-Face Grout Encapsulation in Los Angeles Subway.

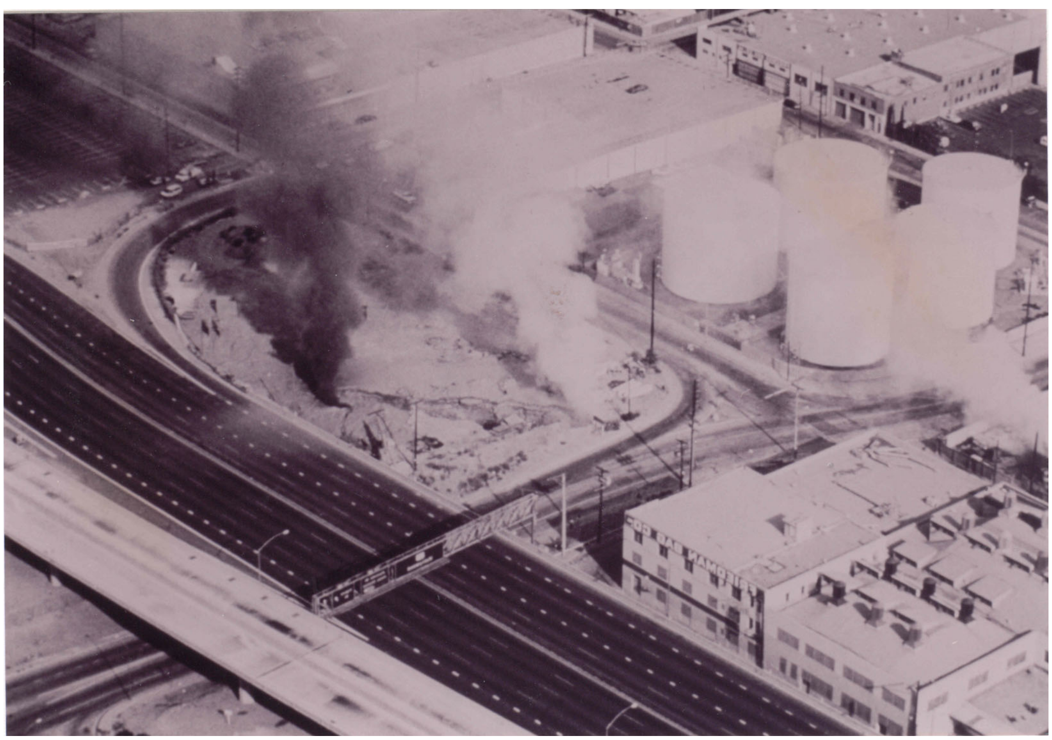


Figure 32. Collapsed Tunnel due to Los Angeles Subway Fire.

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 (62 ft.) 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.

Grouting was not permitted while the Metro rail traffic 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 (7/8-in.) 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 (6 in.) 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 (TACSS) 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 grain size distribution. Alternatively, such LMGs can be injected into voids in rock masses as a bulk infill material or as a component in a seepage remediation program.

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 33) or sinkholes, and for structural underpinning. Compaction grout can also be used to seal off major water ingresses through open channel systems.

b. Design Considerations

Figure 28 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 coarser 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)

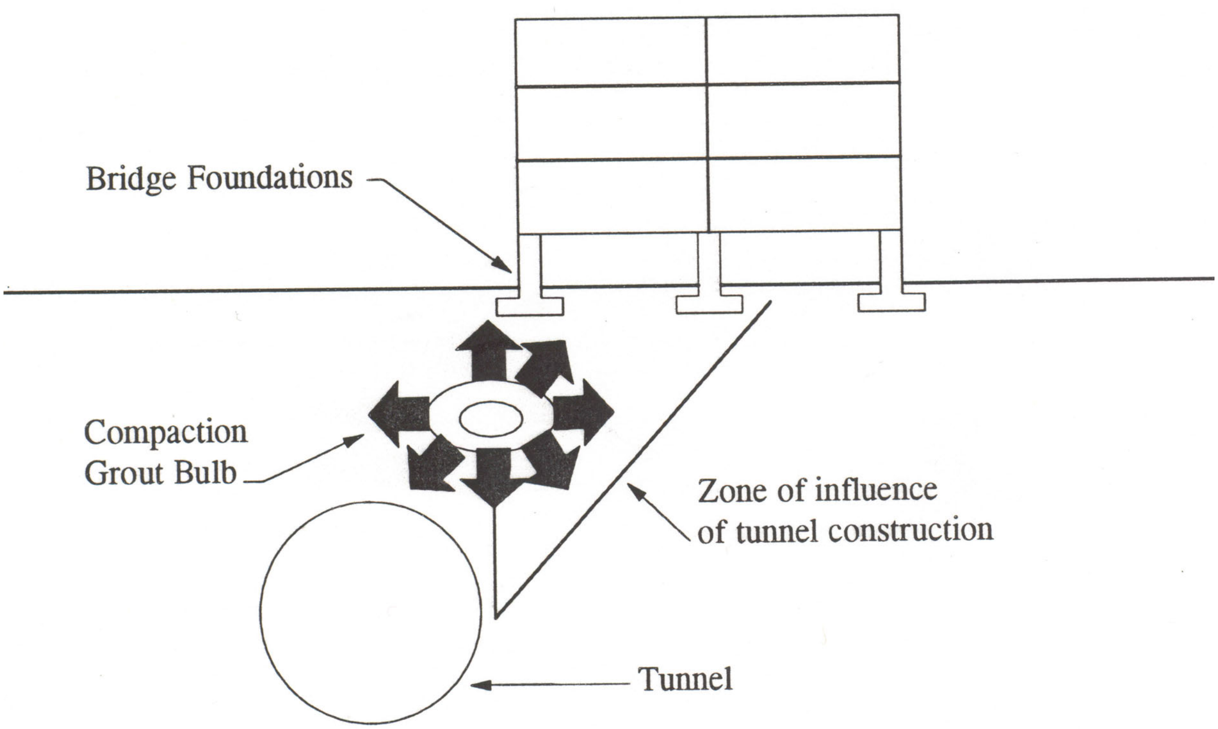


Figure 33. Prevention of Tunnel-Induced Settlements using Compaction Grouting.

For compaction grouts used for void fill or for water cut off, different rheological properties may be preferable, e.g., a slightly higher acceptable slump (up to 75 mm (3 in.)) or the use of polypropylene fibers in the mix.

As in all specialty geotechnical processes, the input of a specialty contractor should be sought beginning with 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 tunneling. There are no mathematical equations 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 (e.g., Ground Penetrating Radar, electrical resistivity, electromagnetic imaging, tomography) can provide more far-ranging data than the point-specific information from a single borehole.

c. Design of Grouting Programs

- **Spacing:** For compaction grouting for densification or redensification, grout pipes are normally installed at 2.4 – 4.6 m (8 – 15 ft.) intervals for tunneling projects, 2.0 – 5.0 m (6.6 – 16 ft.) intervals for site improvement, and 1.0 – 3.0 m (3.3 – 10 ft.) 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 – 9 m (10 – 30 ft.). In such instances, tertiary holes are usually required to ensure and verify satisfactory performance. The pipe diameter should be at least 76 mm (3 in.) 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 – 12% of the volume of soil being treated, whereas for void filling, individual stages may consume tens 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% and not more than 25%. 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 (1 in.) slump, as measured at the header. Depending on the application, other additives to the mix may include gravel, coarse sand, fibers, or anti-washout agents. Similarly, in certain cases, e.g., sinkhole remediation in a dam core, no cement is added to assure that no "hard point" is created in the dam.

d. Construction Equipment

Mixers and Pumps

The low mobility grout will require different mixing, pumping, and delivery equipment than more fluid 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 continue 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 ready-mix 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 that exceed criteria upon delivery, or at any time during the pumping operation from that batch.

In general, the contractor has more control over the properties and consistency of the grout when he batches it on site. In addition, site batching can limit wasteage of materials and delays.

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 – 1m (1 – 3 ft.).

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. As a result, compaction grouting costs vary from as low as \$5.00 per cubic meter of soil treated to more than \$50.00 per cubic meter (\$4 - \$40/yd^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 – \$120 per cubic meter (\$45-\$90/yd^3).

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 – 200 mm (6 – 8 in.) 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 – 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 (23 ft.) 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 m^3 (580 yd^3) 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 (0.2 in.), as compared to the more than 350 mm (13.8 in.) of settlement that had occurred before grouting was implemented.

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

The Worthington Road Bridge, which crosses the New River in Imperial County, California, 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. Groundwater was present in all boring locations, at depths of approximately 2.7 m (9 ft.).

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 (300 ft.) west of the west abutment and 76.2 m (250 ft.) east of the east abutment. The grouting contractor drilled grout holes to firm stratum at a depth of 9.1 m (30 ft.) 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 8 – 9%, verifying the effectiveness of the compaction grouting program.

3.5 JET GROUTING

The jet grouting technique employs high pressure, high velocity erosive jets of water and/or grout to break down the soil structure, removing varying proportions of soil and replacing them with a cement-based grout. The result, when set, is usually termed soilcrete. 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 generally less permeable and stronger 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, by increasing the strength of the weaker soils.

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 an excavation support, it can have a considerable economic advantage. Jet grouting is a comparatively 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 the underpinning units.

Scour Protection

When used as an underpinning system, jet grouting has proven to be an effective means of providing scour protection around bridge piers and marine works.

Excavation Support

Jet grout can be conducted 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. Steel reinforcement can be placed in the soilcrete to enhance axial and lateral capacity.

Liquefaction Mitigation

Jet grouting transforms potentially liquefiable soils into a cemented mass.

Treatment of Karst

Jet grouting has been used to remove clay from karstic features and replace it with engineered grout.[60]

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 (435 psi)) 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 Table 3 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 occasionally be lower due to the air entrainment, although the details of the individual site will govern. 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 (coarser soils give higher strength), and the various operational parameters that 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. Allowable design values should be restricted to ≤ 50% f_c anticipated, to accommodate inherent soilcrete variability.

Program Design

- **Spacing:** Jet grouted columns can be in the range of 0.8 m – 4 m (2.6 – 13 ft.) in diameter, and interconnected overlapping columns are constructed in continuous rows in a primary/secondary sequence.

Table 3. Range of Typical Soilcrete Strengths (Three-Fluid System).

Soil Type	Soilcrete Unconfined Compressive Strength, kPa (psi)	
Clean Sands & Gravel	6,900 – 14,000	(1,000 – 2,000)
Silty Sands	4,000 – 10,000	(600 – 1,450)
Sandy Silts & Clayey Sands	2,400 – 6,900	(350 – 1,000)
Silts and Low Plasticity Clays	2,100 – 6,900	(300 – 1,000)

- **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 requirements depend 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, although values as low as 0.5 have been used. Bentonite and other materials, including additives, may be used, depending on the specific project, but should all be subject to the engineer's approval and satisfactory testing.

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, or with strict observational inspection.

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 up to 50 MPa (7200 psi). 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 monitored and controlled.

Jet Grouting Systems

The choice of grouting system and operational parameters is typically left to the contractor, but is 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 – 0.8 m) and strongest soilcrete in granular soils. The double-fluid system produces larger diameter columns of broadly similar strength, while the triple-jet system produces larger diameter columns (2 – 3 m) of higher strength.

Note that since this manual was first drafted, “Superjet” grouting and “Cross Jet” grouting have both been introduced into North American practice. The interested reader is referred to the Proceedings of the GeoInstitute Grouting Conference (2003), and of the ADSC Conference (2004).

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, Double	Triple, Double
Clays	Triple	Triple

d. Cost

Jet grouting is designed to solve unique problems normally untreatable by other methods. The cost of 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 (\$150/yd^3) of ground treated, and a typical range is \$150 – \$300 per cubic meter (\$115 – \$230/yd^3).

Table 5 presents jet grouting prices for underpinning and excavation support, based on evaluation of more than 65 projects completed in the United States. The costs shown include mobilization, testing, and demobilization, which ranged from \$25,000 – \$50,000. These items are project specific and will vary depending on project size, but typically would represent 5 – 15% 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.

Table 5. Range of jet grouting prices.

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

NOTE: Soilcrete pricing includes mobilization, testing, and demobilization.

e. Advantages and Disadvantages

Advantages and disadvantages of different jet grouting systems are summarized in Table 6.

Table 6. Jet grouting system advantages and disadvantages.

System	Advantages	Disadvantages
Single-Fluid	- Simplest system and equipment - Good to seal vertical joints - Good in cohesionless soil4	- Smallest geometry created - Hardest to control heave - Difficult to control quality in cohesive soils
Double-Fluid	- Most utilized system - Availability of equipment and tooling - High energy, good geometry achieved - Most experience - Often most economical	- Very difficult to control heave in cohesive soils - Spoil handling can be difficult - Not usually considered for underpinning
Triple-Fluid	- Most controllable system - Highest quality in difficult soils - Best underpinning system - Easiest to control spoil and heave	- Complex system and equipment - Requires significant experiences
SuperJet	- Lowest cost per volume treated - Best mixing achieved	- Requires special equipment and tooling - Difficult to control heave in cohesive soils - Spoil handling difficult - Cannot work near surface without support - Highest logistical problems
X-Jet	- Confidence of geometry - Controllable materials cost - Best for soft, cohesive soils	- Very specialized equipment that requires daily calibration - Limited experience available

f. 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 (120 miles) east of Phoenix, Arizona. Heavy rains pushed the river's water more than 9.1 m (30 ft.) 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 34. 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 in Figure 35. 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.

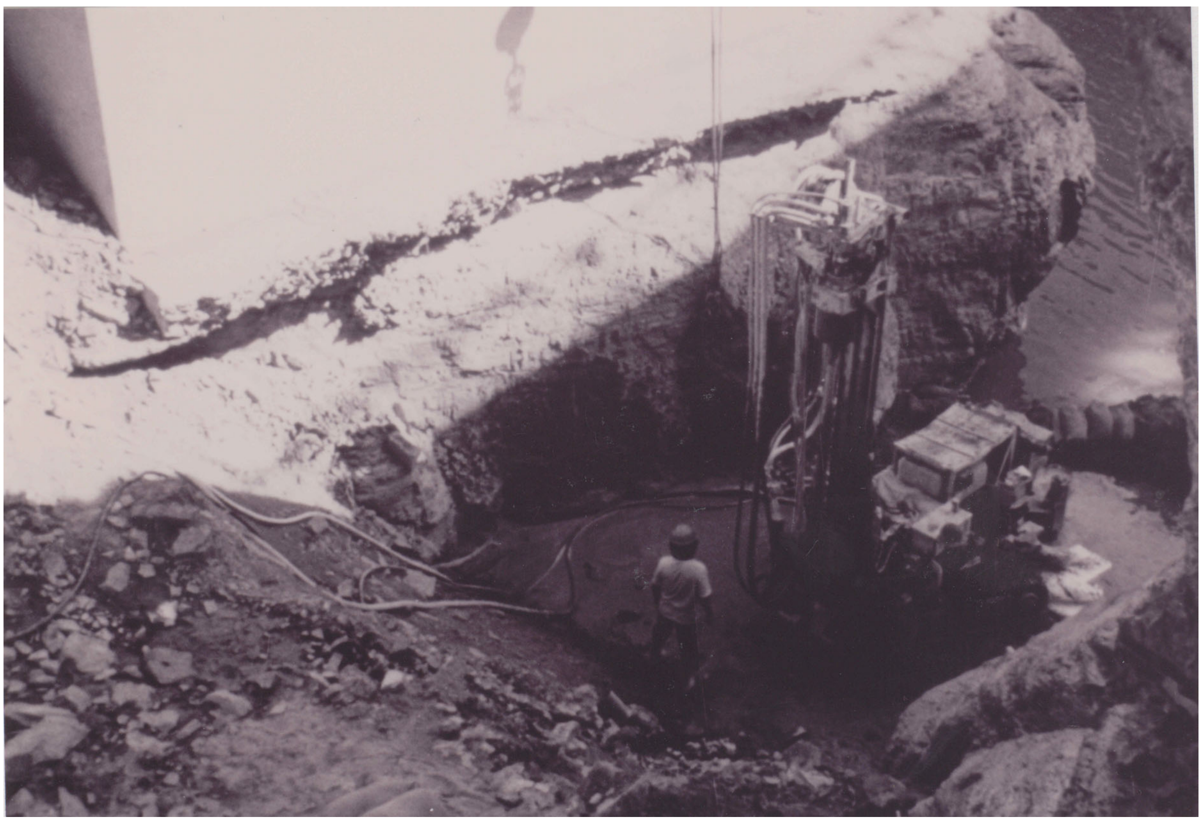


Figure 34. Salt River Bridge Scour.

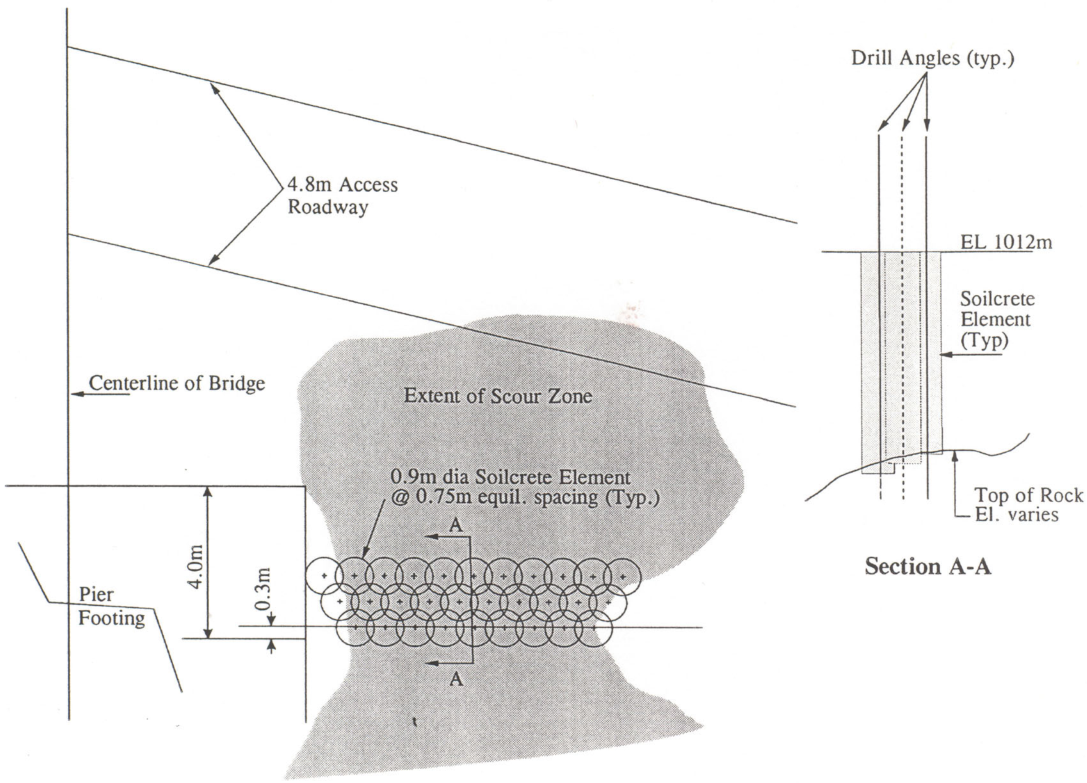


Figure 35. Scour Zone after Stabilization.

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

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 in Figure 36. 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 (19 ft.) below existing grade.

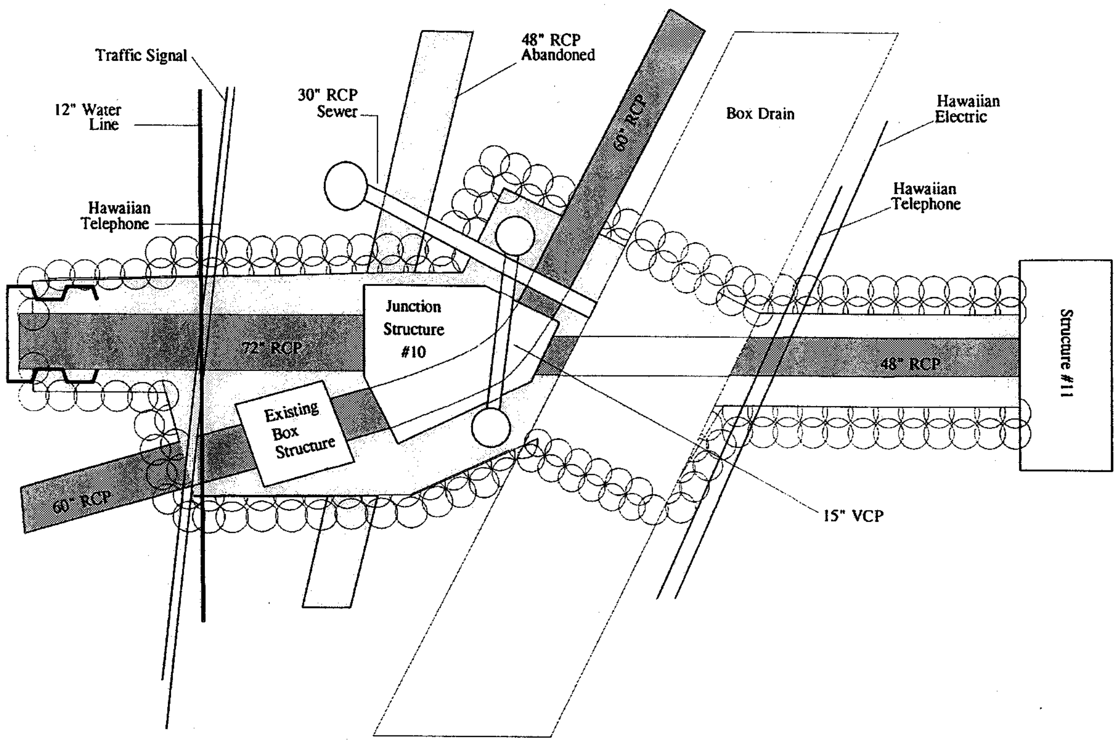


Figure 36. Soilcrete Wall around Utilities.

The soil profile consisted of very loose silty sands to a variable depth of approximately 6.3 meters (21 ft.), where hard coral was encountered. The groundwater table was only 0.6 m (2 ft.) 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 (20 ft.) wide reinforced concrete double box drain, 0.75 m (2.5 ft.), and 1.5 m (5 ft.) 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 in Figure 37. 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.

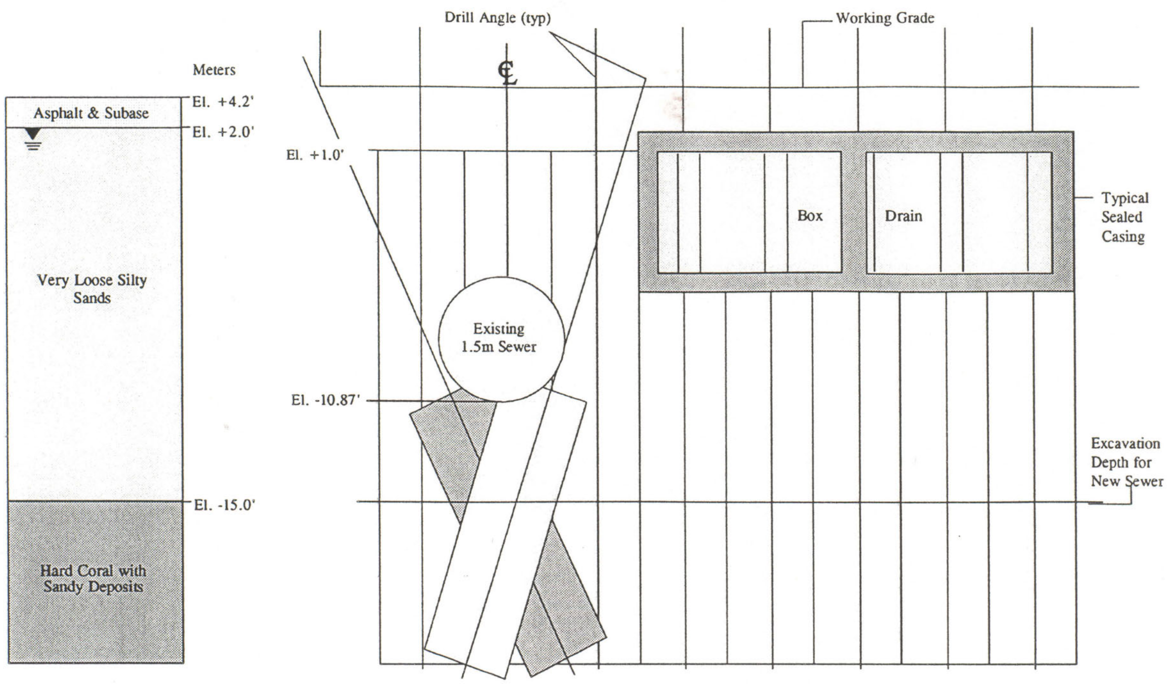


Figure 37. Soilcrete Columns Angled to Protect Existing Utility.

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 it was 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 1-2 m (3

– 6.5 ft.), although most examples involve "fans" of grout holes that 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 tunneling method, when used in compensation grouting. However, individual sleeve volumes of 30-135 liters (8 – 36 gal) 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, fly ash, 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 (940 psi) and pumping rates between 4 – 40 liters (1 – 10.6 gal) 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 still 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.

e. Case History

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 (31 ft.) 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 38). The tunnel was to be driven under shallow (4.5 m (15 ft.) minimum) cover through soft clays, and maximum predicted settlements caused by the tunnel boring machine were as much as 130 mm (5 in.). Since the tunnel was to pass under a number of buildings owned by the Esso Imperial Oil Company (Figure 39), 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 (30 ft.) deep shafts, as shown in Figure 40. The 4.5 m (15 ft.) diameter shafts, as shown in Figure 41 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 (0.016 in.).

Figure 38. Existing Tunnel, Sarnia, Ontario.

Figure 39`. Aerial View of Proposed Tunnel Site.

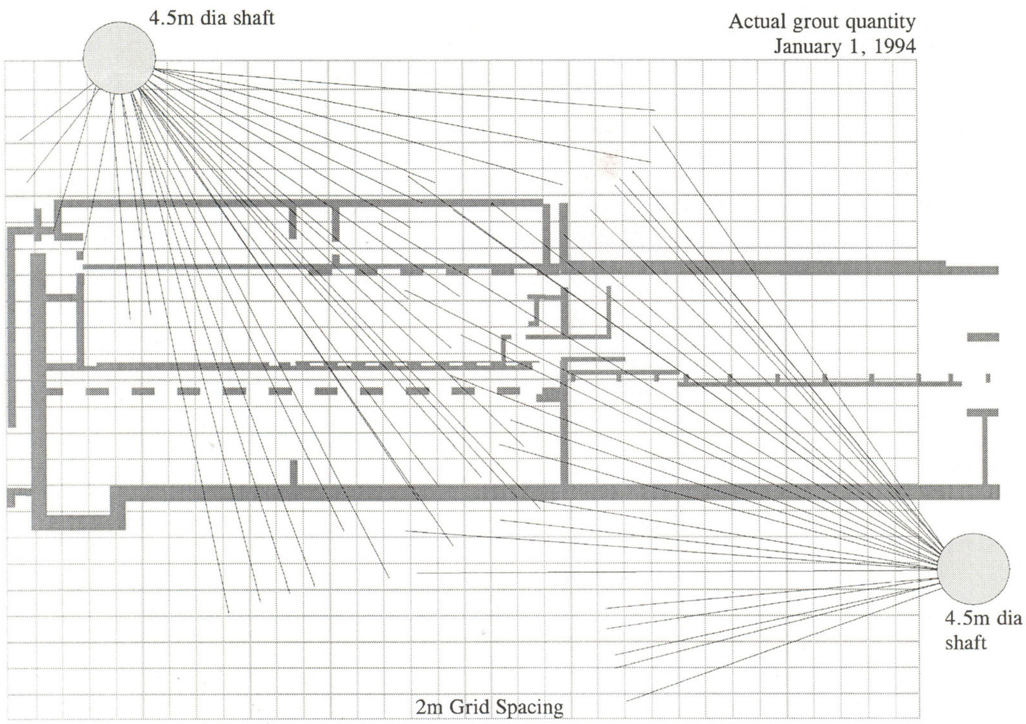


Figure 40. Grout Pipe Locations for Tunnel Site.

As the 9.4 m (30.8 ft.) diameter tunnel boring machine passed within 9 m (30 ft.) 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 (0.25 in.) 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 (2.5 ft.) diameter cast iron service water pipes that were critical to the refinery's operation, an electrical substation, and a river water pump house.

Figure 41. Access Shafts used to Inject Grout.

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, disadvantages, design considerations, program design, 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. Also, active mineral quarrying adjacent to a highway has been known to cause sinkholes and surface depressions in the highway, due to dewatering and/or movement of fines. In such conditions, a remedial grout curtain is essential.

4.2 DESIGN CONSIDERATIONS

Before deciding if grouting is appropriate for a particular site, a thorough subsurface investigation should be conducted. Rock masses can be highly variable, including weak or loose rock, rock with stress fractures, rock with large voids, and rock with open fractures and/or possessing high permeability. Some rock masses may be erodible or soluble.

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 final grouting design, and the associated program cost estimate, can then be logically 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. Often, 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.
- premature plugging of fissures by thickening the mix too quickly, by unsuitable injection methods or formulations, or by using inappropriate drilling and flushing techniques.
- improper hole spacing or orientation of grout holes.

These can be rectified by utilizing knowledgeable and experienced personnel to design, construct, supervise, inspect, and control 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 rock fill dams, concrete dams, tunnels, mines, quarries, deep excavations, 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 overall 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 and responsive modifications 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 two 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 from the surface or from the face 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, to control water, 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 specific residual permeability, as measured by post-grouting tests, or an increase in rock mass strength or homogeneity, as illustrated by core-sample testing, load testing, or cross-hole seismics.

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, inclination and diameter, quantities of grout, grouting equipment methods and parameters, and developing performance and verification 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] Whereas traditional practice was based on neat cement grouts, current practice features the use of suites of multi-components, balanced formulations with carefully controlled fluid and set properties[58].

c. Performance Monitoring

Detailed performance monitoring and evaluation is an integral part of the grouting program. Real-time evaluation of 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 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.[46,47,58]

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 during service. 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, although the use of water flush is strongly recommended for fissure grouting. 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 and, indeed, should be used to determine the latter. 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 42; and
- upstage (ascending stage), as shown in Figure 43.

The advantages and disadvantages of upstage and downstage methods are summarized in Table 7. 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. It is not unusual to find that the uppermost stage (in typically the poorest rock) must be downstaged, but that the other stages can be upstaged.

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 44.

Regarding grouting pressures, there are various "rules of thumb", summarized in Figures 45 and 46, and ranging from 1 – 4 times the theoretical weight of rock above the injection point. Many factors will dictate the site-specific choice, such as geological and structural conditions; but the maximum safe pressure can be confirmed in preconstruction testing.

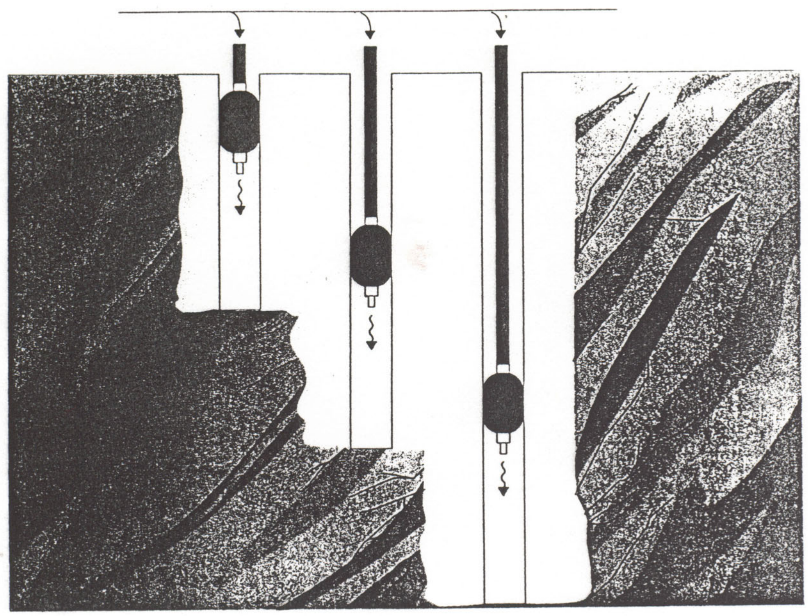


Figure 42. Downstage Grouting.

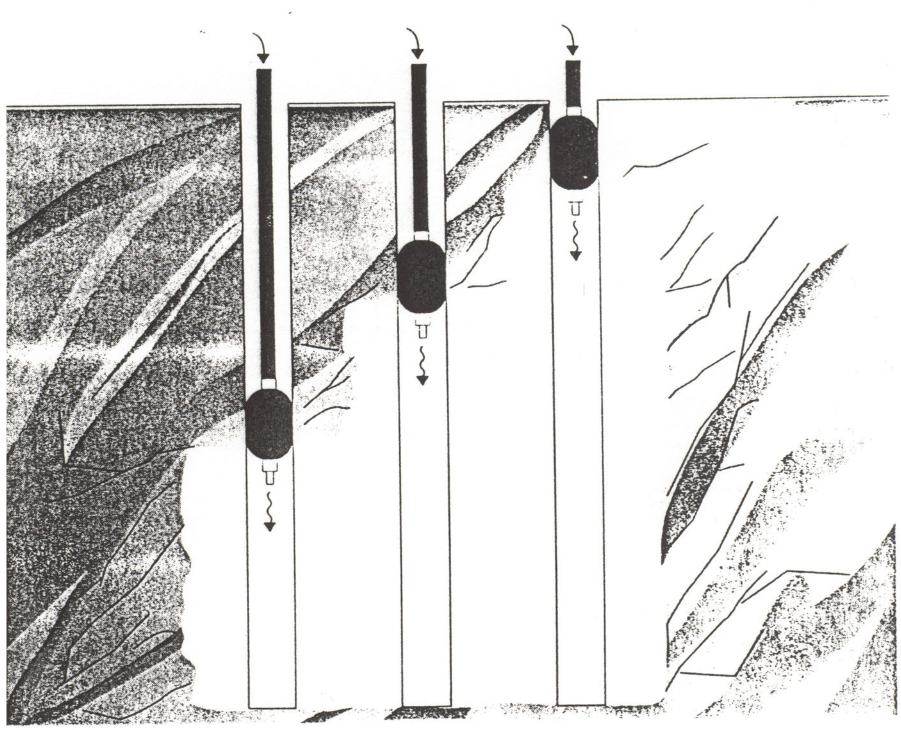


Figure 43. Upstage Grouting.

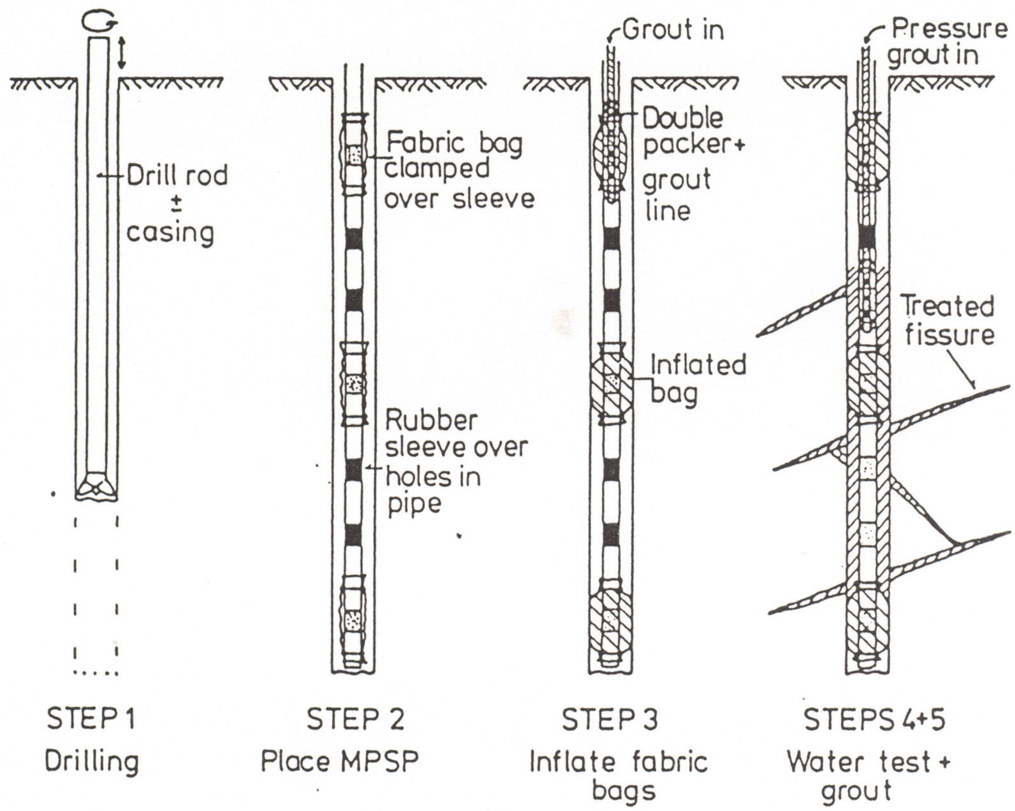


Figure 44. Multiple Packer Sleeve Pipe Process.

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.

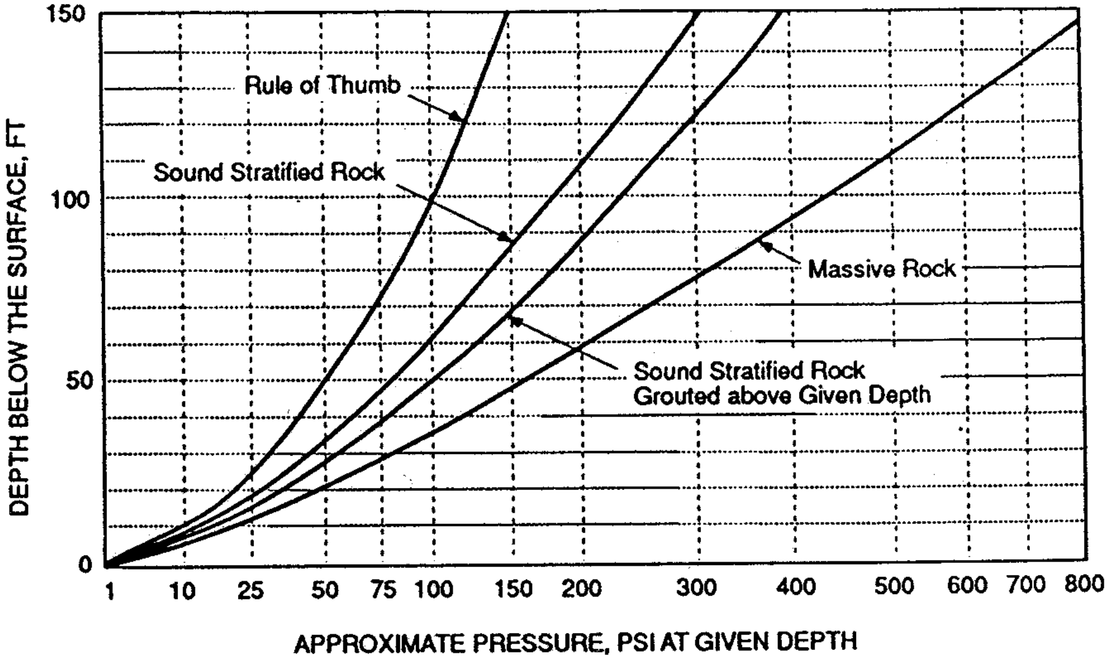
4.5 COST DATA AND BID ITEMS

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 performed under a separate contract.

Table 7. Major advantages and disadvantage of downstage and upstage grouting of rock masses.

	DOWNSTAGE	UPSTAGE
ADVANTAGES	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. 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. Stage length can be adapted to conditions as encountered to allow "special" treatment.	Drilling in one pass. Grouting in one repetitive operation without significant delays. Less wasteful of materials. Permits materials to be varied readily. Easier to control and program. Stage length can be varied to treat "special" zones. Often cheaper, since net drilling output rate is higher.
DISADVANTAGES	Requires repeated moving of drilling rig and redrilling of set grout: therefore, process is discontinuous and may be more time consuming. Relatively wasteful of materials, and so generally restricted to cement-based grouts. May lead to significant hole deviation. Collapsing strata will prevent effective grouting of whole stage, unless circuit grouting method can be deployed. Weathered and/or highly variable strata problematical. Packer may be difficult to seat in such conditions.	Grouted depth predetermined. Hole may collapse before packer introduced or after grouting starts, leading to stuck packers and incomplete treatment. 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. Artesian conditions may pose problems. Weathered and/or highly variable strata problematical.



From: U.S. Army Corps of Engineers (1984)

Figure 45. Injection Pressures used in U.S. Grouting Practice.

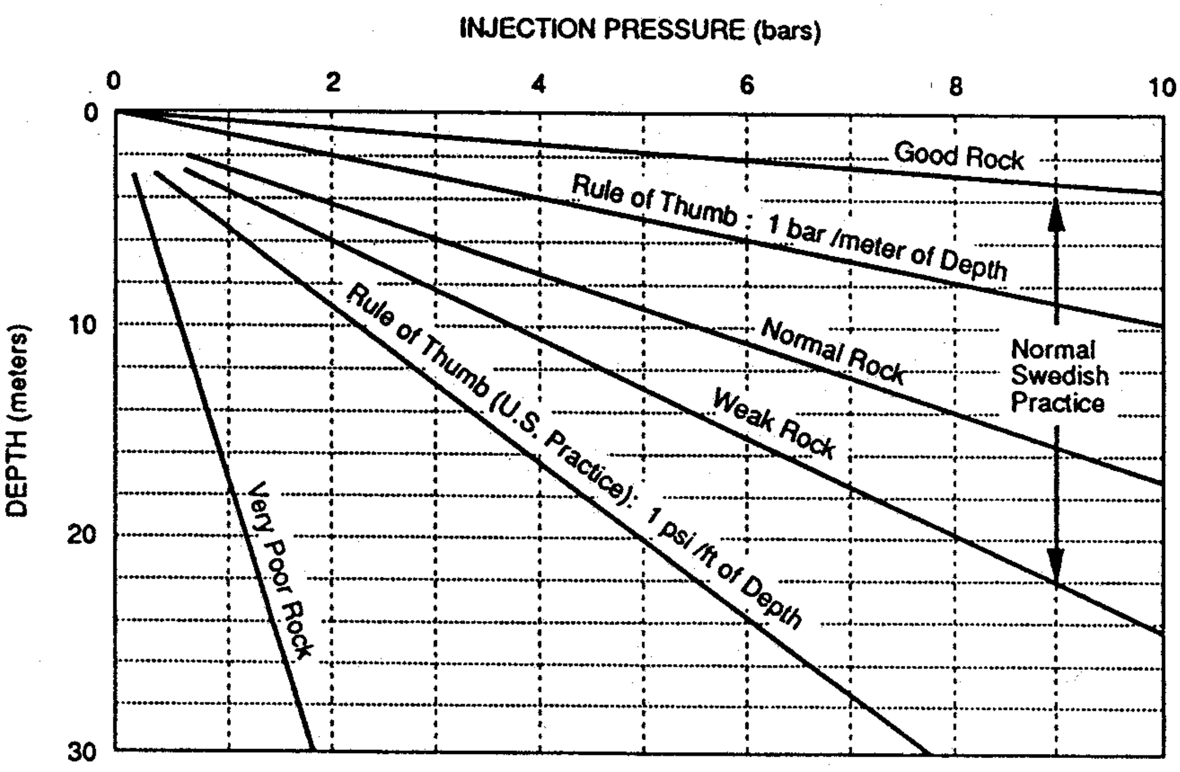


Figure 46. Injection Pressures used in Swedish Grouting Practice.[5]

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 on the basis of the linear meter of holes actually drilled, typically including the cost of washing. In most 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. Redrilling set grout is typically priced at 50% the rate for rock drilling.

Materials are paid on the basis of weight of each component injected into the holes. Grout injection (or placement) is paid on the basis of either the number of cubic meters of grout injected or, preferably, by the pump 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 there from. 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 – This included all instrumentation other than that integral to control or analyze the drilling and grouting data. (The latter data systems can also be priced separately either as a lump sum, or by time.)

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 ultra-fine cement.

The Helms Pumped Water Storage Project, completed in 1982, is a 1,161 MW hydroelectric facility located in the Sierra Nevada Mountains in California. Underground features include 6,700 m (22,000 ft.) of concrete lined pressure tunnel up to 8.2 m (27 ft.) in diameter, 2,130 m (6,990 ft.) of unlined access tunnels up to 9.5 m (31 ft.) in diameter, and three deep vertical shafts, one steeply sloping inclined shaft, and two major chambers more than 300 m (1,000 ft.) underground (Figure 47).

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 48). 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.

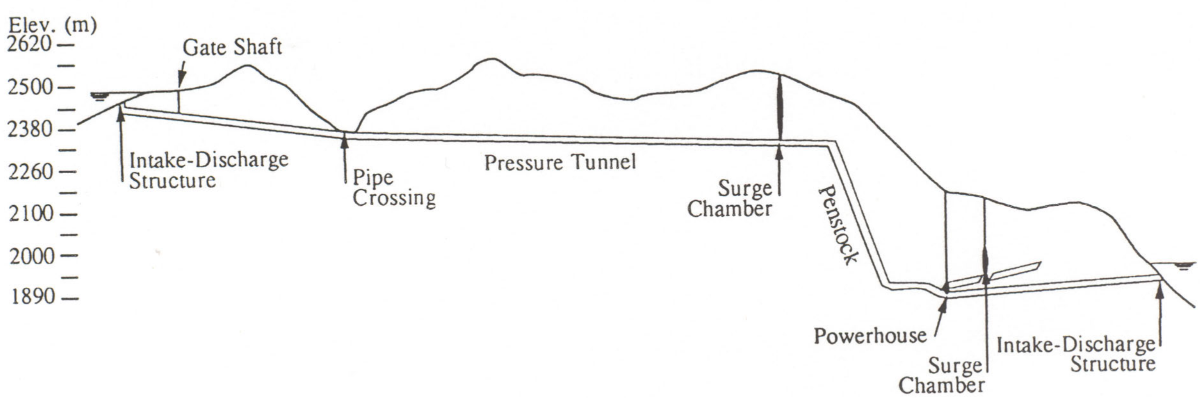


Figure 47. Underground features of the Helms Pumped Water Storage Plant.

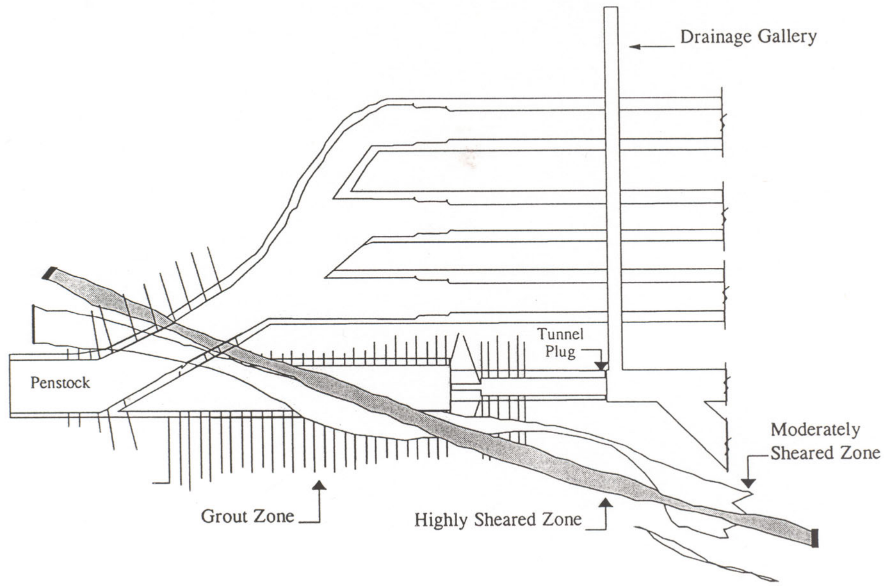


Figure 48. Plan View Showing Location of Shear Zone West of Powerhouse Station.

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.

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, could also cause 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 downstream 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 ultra-fine 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 – 1.5 m (1.0 – 5 ft.), 1.5 m – 4.6 m (5 – 15 ft.), 4.6 m – 7.6 m (15 – 25 ft.), 7.6 m – 12.2 m (25 – 40 ft.). Rings of grout holes were drilled and stage-grouted on a primary, 12 m (40 ft.) spacing along a tunnel section. Secondary, then tertiary, grout rings split the primary spacing for a final spacing of 3 m (10 ft.). 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%, 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%. 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 karstic limestone cavity infill, backfilling of old mineral workings, and repair of scour problems under bridges. Slabjacking is used to lift and relevel concrete slabs and, therefore, involves far lower grout volumes and different construction methodologies.

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 and dissolve over time, thus forming in-situ cavities. These can potentially cause the ground above to collapse or sink if they migrate to the surface. These phenomena are known as sinkholes. LMGs can be injected into these limestone cavities to seal the cavities and redensify the loosened overburden soil. The rheology 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.[50,60]

Drilling and grouting methods are commonly used to fill collapsed or abandoned coal and iron mines to prevent surface subsidence, and this has major application in Ohio, Pennsylvania, New Jersey, 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 can be affected by scour that 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 if it is found that even LMGs are washed out prior to setting in-situ.

b. Slabjacking

Slabjacking is used to correct settlement associated with concrete slabs formed over soils, such as organic soils, compressive clays and silts that have consolidated, or materials that 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, incorporating bentonite, sand, ash and/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 (when no cracking is caused to the slabs) are obtained when the slabjacking is uniformly and gradually conducted. Slabjacking can also be used to "pump" expansion joints that 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 voids. Another problem is containing the grout within the zone to be stabilized, although low slump grout "barriers", accelerated grouts, and grout filled fabric forms have been used to minimize this problem, as illustrated in Figure 49.

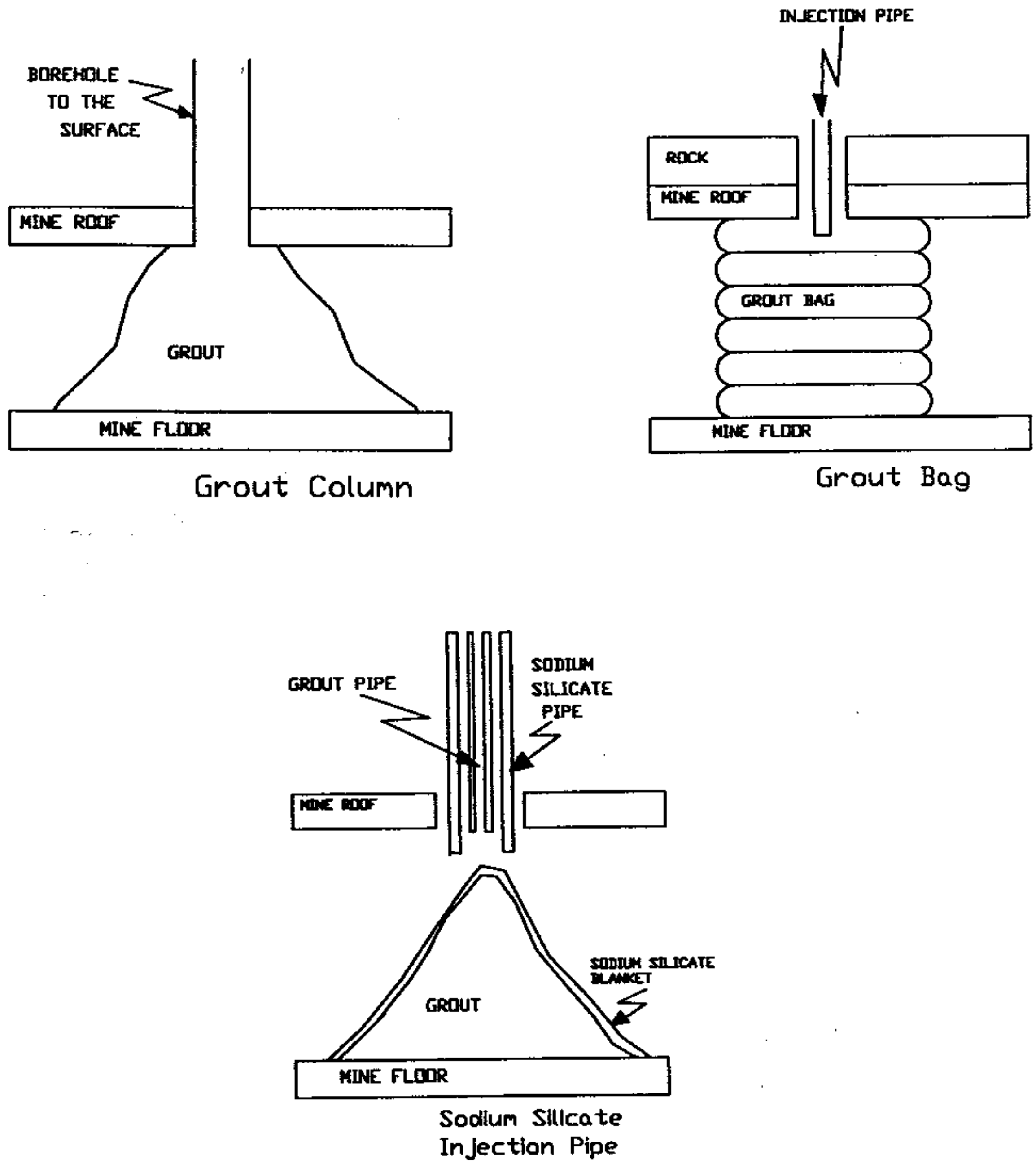


Figure 49. Void Filling Methods.

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 pavement is extended.
- A smoother riding surface is established.

Slabjacking has the following disadvantages:

- Cracks 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 may 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 that 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 may involve intensive exploration drilling, usually supplemented by a variety of geophysical (e.g., GPR, resistivity) and hydrological tests. Sudden variability in ground conditions may be anticipated between adjacent boreholes in karst.

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 groundwater regime, and, in particular, if the water is moving, where it is moving, and 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 more than 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. Clay trapped in grouted karstic cavities can be blown out if subjected to prolonged differential head.

It is important to realize that the extent of a cavity is unknown after penetration by just one grout hole and even the thoughtful implementation of appropriate geophysical techniques may not yield conducive information. 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 0.8 – 28 m^3 (30 – 1,000 ft^3) or more per injection period. A limit may also be placed on the maximum amount of grout to be injected into a single hole. This practice differs from that recommended for 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, or radically different materials and methods should be considered. Additional exploration, consultation, evaluation, and design of treatment can then take place without delaying the project. These measures may call for specialized grouting procedures or materials, such as foaming agents or accelerators, positive cutoff diaphragms or formed concrete wall, hot bitumen, additional excavation, grout filled bags, 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 (10 ft.) or less are not unusual for "tightening" purposes.

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 releveling, 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 – 1.8 m (5 – 6 ft.). 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 (although such applications are remarkably few in number).

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 the classic performance monitoring technique. Appropriate geophysical methods may also be of value.

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. High pressure pumps are not required for injecting HMGs, and conventional LMG pumps are adequate for fast void fill and/or seepage remediation.

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 (400 psi) or less and displacement rates of 0.007 m^3/min (0.004 cfs) are sufficient. However, it is preferred that the pump develop pressures to 10,350 kPa (1500 psi) and displacement rates varying from 0.003 m^3/min (0.0018 cfs) to 0.056 m^3/min (0.033 cfs).

b. Materials

Void Filling

The materials used in a void filling grouting operation can vary from non-cementitious 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 (i.e., LMG, fissure grouting, and HMG).

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 8.

Table 8. 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% - 10%, 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% passing a 2.38 mm (#8) sieve, with not more than 20% finer than 50 microns. 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 – 50% 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 comprises 1) mobilization and demobilization, 2) drilling (production and exploratory), 3) flushing and water testing, 4) mixing and injecting grouts, 5) materials, and 6) verification drilling, and testing. 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 grout hole grid pattern may range from 3 – 30 m (10 – 100 ft.) on center. The diameter of the drill hole normally ranges from 76 mm – 203 mm (3.0 – 8.0 in.) and cost starts at $25 per linear meter ($7.62 per lft.). The cost for supplying, mixing, and injecting the grout normally ranges between $75 – $200 per cubic meter ($57 – $153/yd^3). A review of costs for bridge scour repair using concrete fabric forms from 1968 to 1976 in Pennsylvania indicates a range of $300 – $1,000 per cubic meter ($230 – $765/yd^3).[51]

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 – $100 per square meter ($6.50 – $9.30/ft^2) of slab raised up to 50 mm (2 in.).

5.7 CASE HISTORIES

Case 1 - Mined Out Areas; Rock Springs, Wyoming

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 – 2.4 meter-deep (6 – 8 ft.) voids.

The grouting contractor performed a test program to verify the feasibility of the grouting approach. Pillars, 2.4 m (7.9 ft.) in diameter, were constructed on 6 m (19.7 ft.) 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, Georgia

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 (49 ft.) below the ground surface. The site had not evidenced damage at the time of the trials.

HMG grouting was selected as the preferred grouting technique. Grout holes were placed on a 6 m by 6 m (19.7 ft. x 19.7 ft.) grid pattern, utilizing grout consisting of sand, cement, and bentonite, with a slump of 200 mm (8 in.).

Significant grout takes in adjacent holes prompted the engineers to drill and grout secondary holes. A total of 2,294 m^3 (3,000 yd^3) of grout was injected through 4,880 m (16,000 ft.) of drilled holes.

Initially, grout was injected into the primary holes that filled voids ranging from 1 – 4.5 m (3.3 – 15 ft.) 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, Pennsylvania[51]

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, Pennsylvania. 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 under-scour, it was elected to repair the problem immediately.

The river at this point was approximately 762 m (2,500 ft.) across, which would make the cost of an access road and cofferdam extremely expensive. The pier rested on a rock bottom that 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 50. 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 technique. 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.

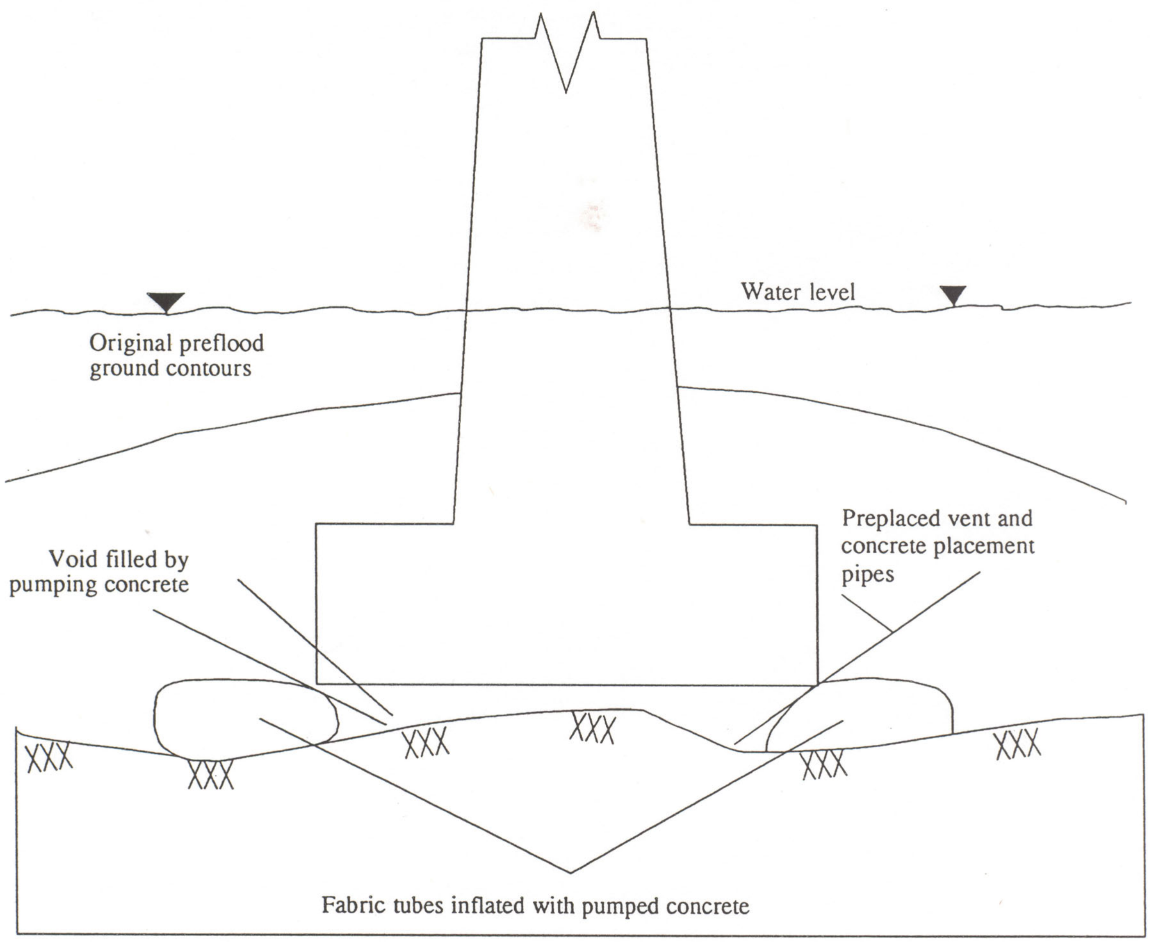


Figure 50. Void Space Filled by Concrete-Filled Fabric Tubes.

CHAPTER 6
BIDDING METHODS AND CONSTRUCTION SPECIFICATIONS

Construction specifications for grouting operations must be specifically tailored to the specific project objectives. Caution should be exercised when attempting to use a preexisting "standard" specification without sensible 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, that they are equally applicable for the proposed project and that they are not mutually contradictory. Conversely, if not very judiciously employed, use of "boiler plate" specifications can cause problems, 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 must never be used directly; they must 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 the 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 timeframe. The prospective bidders should be required to submit the resumes of their key personnel, including the proposed project engineer and superintendent(s) who <u>will</u> be used on the project. 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 resources and experience; and the grouting is sublet to Subcontractors specializing in this type of work, except in the case of tunnel grouting, where the tendency of tunneling contractors 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 Quantities

- <u>Test Grouting</u>. For medium and large projects, probably the most reliable method for estimating quantities is to conduct a test-grouting program, preferably during the design stage. The site chosen for testing should be geologically representative of what was found during subsurface exploration; and the means, methods, and materials must be substantially those envisaged for the production work.

- <u>Evaluation of Subsurface Information</u>. 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. The local knowledge held by 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, regardless of whether work under the other items of the program is performed.

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, Grout

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.

Grout Injection Parameter Recording and Analysis

If a project warrants a high degree of real time parameter monitoring and analysis, then the computer based system should be paid for separately, either as a lump sum or as a weekly or monthly recurrent fixed cost.

6.2 SOIL GROUTING SPECIFICATIONS

a. 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) selection of the types and relative amounts of various types of grout; 4) proper operation of the grout mixing and injection system in harmony with the actual ground response; 5) continuous recording (preferably automatic), graphical display and analysis of the injection data; and 6) 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 permeation grouting using sodium silicate 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 a maximum residual permeability 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.

[Note: In both cases, the uniformity of the treatment will reflect the heterogeneity of the soil.]

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 (5 ft.). The grout pipes should be of the sleeve-port type, with grout ports at 0.3 – 0.5 m (1.0 – 1.6 ft.) 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 alternative methods, such as batch mixing; they must 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 grout flow pressure for any one sleeve should not be more than 30 kPa per meter (1.0 psi per ft.) 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 75 kPa per meter (3.3 psi per ft.) 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 one 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 – 50 minutes, with most grout having gel times in the range of 15 – 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 at least 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. Grout components (such as sodium silicate, reagent, and 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. These items 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.

Contractor's 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 Contractor's 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 (0.8 in.) or less, but in no case exceeding 38 mm (1.5 in.), 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's supervision to ensure 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 ready-mix 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 (1000 psi) as measured at the point of injection, and a pressure of up to 13.8 MPa (2000 psi) at the pump. The pump shall have a minimum injection capacity of 4 liters/minute (1 gpm) and a maximum up to 140 liters/minute (37 gpm).

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 (2 in.). 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 (2.0 in.) 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 (1.0 ft.) 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 Contractor's 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 (0.75 in.) or less but in no case exceeding 38 mm (1.5 in.) (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 Contractor's 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 the 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 an owner's representative or the 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

A. 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.

B. The Contractor shall be responsible for monitoring of structures during the Compaction Grouting operation. The monitoring shall detect movements greater than 3 mm (0.125 in.). 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 (0.125 in.) 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 the duration of the program to give continuous, visible indication of movement of structural members.
   d. A manometer system 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.

C. The Engineer shall monitor the grouting operation. The monitoring shall include review and approval of mix design, slump tests, strength tests, injection pressures and volumes, and the Contractor's grout logs.

D. 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 – 1 m stages).
   c. The depth of each stage. (0.3 – 1 m 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.

E. 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 the owner's 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 that 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 approved by the Engineer, has been injected in a given stage.
c. Pressure, as measured at the probe header, reaches
    i. pressure determined by the engineer to be adequate.
    ii. 4.2 MPa (600 psi) or greater, when injection probe is less than 15 m (50 ft.) from surface grades (including face of slopes).
    iii. 5.5 MPa (800 psi) or greater, when injection probe is more than 15 m (50 ft.) from grade.
    iv. spike pressures of 10 MPa ((1450 psi) which shall be considered refusal of initial take.

7. The rate of injection of the grout shall not exceed 60 liters/minute (16 gpm) 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 as defined in 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 the 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 soilcrete 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 soilcrete elements conform to requirements stated herein.

B. The jet grouting method shall be used to install soilcrete in areas indicated on the Drawings to form complete and continuous soilcrete elements. In using jet grouting, the achievement of full, intimate filling with soilcrete between adjacent soilcrete structures is a paramount requirement. Soilcrete 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 soilcrete location data.

3. Jet Grouting - The process of injecting cement grout to create an in-situ soilcrete. The cement grout is injected and mixed with the soil at high velocity through nozzles at the end of a monitor inserted in a borehole. The monitor is rotated at a slow, smooth, constant speed, which, when combined with the rate of withdrawal, achieves a continuous geometry and quality of soilcrete. *(Note: It is often the case that the type of jet grouting, i.e., one-, two-, or three-fluid can be specified by the owner depending on the project goals).*

4. Soilcrete - Homogeneous mixture of cement grout and in-situ soils. (Note: homogeneity must be defined on a project-specific basis.)

5. Soilcrete Element - A column of soilcrete formed by jet grouting that results in a homogeneous mixture of cement grout and in-situ soils.

6. Soilcrete 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 that are discharged above ground surface or mudline during, or as a result of, jet grouting. It is essential that the Contractor's means and methods ensure a full, uninterrupted flow of spoils to the surface at all times during jetting.

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, woodpiles, metal, abandoned foundations, 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, clay stones, and sand and gravel layers may be encountered within the subsurface soils, and will not be considered as either known or unknown obstructions. *(Note: This should be included only if they are known to exist.)*

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 soilcrete 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 soilcrete elements. Acceptance of soilcrete shall require documentation that the soilcrete 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 (and residual permeability, if important).

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 demonstrates 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 soilcrete 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 soilcrete 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 maintain 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. Soilcrete elements shall be numbered and dimensioned to survey baseline established by the Contractor.

4. Mix design and mix procedures of soilcrete installed by jet grouting including cement type, water-cement ratio by weight, and estimated minimum 56-day compressive strength. Also submit proposed soilcrete 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 soilcrete in the event unknown obstructions are encountered.

6. Means and methods to ensure a continuous and uninterrupted return of spoils during jetting.

7. 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 soilcrete 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 soilcrete samples. Catalog cuts or shop fabrication drawings of the soilcrete 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, spoil return, and other related aspects of the jet grouting process.

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

8. 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 soilcrete element, submit to the Engineer

1. Deviations of the center coordinate and diameter from the layout plan to the nearest 13 mm (0.5 in.) 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 soilcrete elements to the nearest 20 mm (0.75 in.).

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 soilcrete installed during the work shift and all soilcrete 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 soilcrete 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 soilcrete 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 soilcrete element, advancement rates of the jet grouting equipment, 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 soilcrete 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 of 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 soilcrete or excavation and backfill performed to overcome the obstruction. A plan of suitable scale to clearly show the details and limits of the soilcrete 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 that 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 soilcrete 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 soilcrete samples, including wet grab and core samples, and provide them to the Engineer. The engineer will form, preserve, cure, transport and test the soilcrete samples, and report the test results. The Contractor shall cooperate with the Engineer and coordinate sampling activities on 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 documention all obstructions and the disposition of how each obstruction was overcome.

D. Wet Grab Soilcrete 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 soilcrete 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 soilcrete 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 soilcrete samples shall be capable of obtaining a discrete fluid sample of soilcrete at a pre-determined depth and shall be capable of accepting particles not thoroughly mixed that are up to 150 mm (6 in.) in any dimension. The sampler shall be lowered empty, air only, to the required depth in the soilcrete, element and then opened. Once filled with the soilcrete the sampler shall be closed to exclude entry or loss of soilcrete and be expeditiously raised to ground surface.

3. Each retrieved soilcrete sample shall be of sufficient volume to produce a minimum of four full cylinders, 75 mm (3 in.) diameter by 150 (6 in.) mm height. All samples

shall be separated and retained from each depth, and immediately provided to the Engineer. The Engineer will then cut all retrieved particles of soil larger than 10 mm (0.4 in.) into smaller pieces that pass a 5 mm (0.2 in.) sieve, and then form immediately the four cylinders of material passing through a 5 mm (0.2 in.) sieve.

4. Soilcrete samples shall be protected from freezing and extreme weather conditions that could have a deleterious effect, at all times in accordance with AASHTO T 23.

5. Soilcrete 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 soilcrete, in the designated soilcrete element, the Contractor shall obtain a full suite of wet grab samples from the next soilcrete installed by that rig. Wet grab samples in subsequent soilcrete elements will continue to be taken until a full suite is obtained.

(Note: In one-fluid grouting, the viscosity of the soilcrete upon attempting to sample may defeat representative wet grab sampling.)

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, permeability, and composition of the soilcrete. Coring of soilcrete shall be performed with a triple tube core barrel, with a face 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 soilcrete. The samples shall be obtained using a PQ-size triple core barrel with a face discharge.

3. Immediately after retrieving the soilcrete 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 85% recovery, or less than 40% RQD, or fewer than two intact cores of length more than 6 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, unless there are convincing technical arguments to the contrary. It must be noted, however, that gravelly or cobbly soilcrete is very difficult to core and may not provide good samples for laboratory testing.

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 *(Unnecessary for holes less than 50 feet deep)*

1. The Contractor shall obtain vertical alignment profiles over the length of one soilcrete 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 soilcrete element exceeds allowable vertical element tolerances, the vertical alignment of the next soilcrete 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 to 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 fly ash may be allowed with the acceptance of the Engineer and submission by the Contractor of test data that confirms no deleterious impact to the soilcrete.

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 that 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%.

B. Water used in jet grouting, grout mixing 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 soilcrete mixing, unless requested in writing by the Contractor and accepted in writing by the Engineer.

2.02 Soilcrete by Jet Grouting

A. Soilcrete 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 soilcrete 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. Soilcrete 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. Soilcrete shall achieve a 28-day *(or 56-day)* unconfined compressive strength of f'c equal to *(fill in)*. Variation in soilcrete quality is to be expected: up to (*fill in*)% of the test results are permitted below the target f_c.

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

D. The total unit weight of soilcrete 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 soilcrete 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 soilcrete uniformity criteria will be determined by the Engineer by evaluation of core samples. The soilcrete shall contain soil fragments with a maximum dimension not to exceed 1/4 of the diameter of the auger, or 300 mm (12 in.), whichever is smaller. In addition, 70% 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.*

F. Permeability of soilcrete shall not exceed (fill in).

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 (0.04 in.) and 6.5 mm (0.25 in.) in diameter. The jet grouting equipment shall be capable of providing at least 48 MPa (7000 psi) 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, if necessary.

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 soilcrete 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 fluid 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 soilcrete 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. *(Note: This is often not a realistic request.)*

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 remediating or 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, 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 Soilcrete

A. Install soilcrete 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. Soilcrete 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% of the jet grout column diameter and shall include allowance for tolerable vertical alignment deviation.

C. Soilcrete shall penetrate to approximately *(fill in project specific requirement)*. Final grouting of the soilcrete elements shall not commence until the Engineer has approved the bottom depth of the soilcrete element. Non-conforming elements shall be replaced at no additional cost to the Department. Where developed, soilcrete 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 soilcrete elements, the Contractor shall obtain samples of in-situ soilcrete in accordance with the locations and frequencies required in the Quality Assurance/Quality Control Program.

F. Any soilcrete element that 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 soilcrete element is completed.

H. Soilcrete elements shall not be installed within 600 mm (24 in.) as measured between outside edges of soilcrete elements that are less than 12 hours old. The 12-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 soilcrete elements or the ground.

3.03 Horizontal and Vertical Alignment Tolerances

A. The maximum horizontal deviation of the as-installed center of any soilcrete element at the ground surface or mudline installation level shall not exceed 75 mm (3 in.) from the layout center coordinate, shown on the accepted Contractor's submittal.

B. The vertical alignment of soilcrete elements shall not deviate in any direction more than 2% from vertical.

C. At the direction of the Engineer, any soilcrete element that exceeds the allowable horizontal or vertical alignment tolerances shall be re-mixed within two days of initial placement, or supplemented with one or more adjacent or overlapping elements, at no additional cost to the Department.

3.04 Obstructions

(Note: This is potentially very important. If they can be quantified in advance, the Contractor can price the risk.)

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 jet grouting, the Contractor shall remove the obstruction or install additional soilcrete 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 continuous during operations, and 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 soilcrete 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 soilcrete mixing operations.

4.01 Method of Measurement

(Note: This applies to relatively straightforward cases where the volume of soil to be grouted is well defined. Other alternatives should be considered in complex situations.)

A. Installation of soilcrete by jet grouting will be measured per cubic meter to the nearest cubic meter, within only the “neat” plan area of the proposed soilcrete 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 soilcrete.

Additional quantities of soilcrete installed to overcome unknown obstructions shall be incorporated into the total measured quantity of soilcrete, as accepted by the Engineer.

Quantities of soilcrete installed by the Contractor during remixing to achieve the performance requirements or that are outside the limits of soilcrete 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 soilcrete 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 soilcrete 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

Installation of Jet Grout	Cubic Meter of Treated Soil

d. Soil Fracture Grouting

Since relatively few projects have been completed in North America to date, it is not possible to provide a very detailed guide specification at this time. The following highlights areas that must 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.

- Key personnel should include a project manager and project superintendent who have provided full-time, on-site supervision of at least one soil fracture project.

- Key personnel must 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 or jacking system to penetrate the formation without causing caving, collapse, or heave. The drilling procedures used for grout pipe installation should be suitable to penetrate all known ground conditions, while ensuring drill hole stability and minimal deviation.

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 or a computer controlled, motor-driven total station.

Precondition Grouting

Precondition grouting is the initial grout injection used to tighten the soils before the need for reactive compensation grouting in is used 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 (1 in.) 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.

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 for 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. In either case, it is essential that the performance goal of the project is clearly stated (e.g., a target residual permeability and/or an acceptable magnitude of residual seepage).

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 must be addressed.

Drilling Equipment

Rotary or rotary percussive methods with water flush may be permitted generally. Few circumstances, other than access problems, or the need to drill exploratory or verification holes, justify use of diamond drilling of grout holes. This method 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. Exclusive or predominant air flush is not acceptable.) 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 or significant hole instability 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 must be specified. Use of progressive cavity pumps should not be an absolute requirement. Automatic recording and analysis 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, bleed, 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 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, washing the pump system during grouting, determining the need for additional grout holes, treating 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 to allow grout to be injected and to prevent later erosion of windows in the rock fractures grout curtain, 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 should 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 multi-pressure 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), except that water flush drilling is not mandatory, and automatic recording and analysis of all grouting and water testing activities is not typically required or necessary.

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 9.

Table 9. 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 (1 ft.) into the lower bag or tube and protrude 0.3 m (1 ft.) 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 (290 psi) 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 – 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 – 26 seconds. A more fluid mix having a flow cone time of 9 – 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 (1/8 in.). 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 each 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 removal of the plugs, all spillage and leakage around the holes 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; 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 cost-plus through a lump-sum-method.

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Technical Summary # 11:

REINFORCED EMBANKMENTS ON SOFT FOUNDATIONS

Excerpt from:

Geosynthetic Design & Construction Guidelines, FHWA NHI-06-116 (DRAFT)

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7.0 REINFORCED EMBANKMENTS ON SOFT FOUNDATIONS

7.1 BACKGROUND

Embankments constructed on soft foundation soils have a tendency to spread laterally because of horizontal earth pressures acting within the embankment. These earth pressures cause horizontal shear stresses at the base of the embankment that must be resisted by the foundation soil. If the foundation soil does not have adequate shear resistance, failure can result. Properly designed horizontal layers of high-strength geotextiles or geogrids can provide reinforcement, which increase stability and prevent such failures. Both materials can be used equally well, provided they have the requisite design properties. There are some differences in how they are installed, especially with respect to seaming and field workability. Also, at some very soft sites, especially where there is no root mat or vegetative layer, geogrids may require a lightweight geotextile separator to provide filtration and prevent contamination of the first lift if it is an open-graded or similar type soil. A geotextile is not required beneath the first lift if it is sand, which meets soil filtration criteria.

The reinforcement may also reduce horizontal and vertical displacements of the underlying soil and thus reduce differential settlement. *It should be noted that the reinforcement will not reduce the magnitude of long-term consolidation or secondary settlement of the embankment.*

The use of reinforcement in embankment construction may allow for:

- an increase in the design factor of safety;
- an increase in the height of the embankment;
- a reduction in embankment displacements during construction, thus reducing fill requirements; and
- an improvement in embankment performance due to increased uniformity of post-construction settlement.

This chapter assumes that all the common foundation treatment alternatives for the stabilization of embankments on soft or problem foundation soils have been carefully considered during the preliminary design phase. Holtz (1989) discusses these treatment alternatives and provides guidance about when embankment reinforcement is feasible. In some situations, the most economical final design may be some combination of a conventional foundation treatment alternative together with geosynthetic reinforcement. Examples include preloading and stage construction with prefabricated (*wick*) vertical drains, the use of stabilizing berms, lightweight fill or column supported embankments - each used with geosynthetic reinforcement at the base of the embankment. In addition to the information in Chapter 2 on prefabricated drains and Section 7.12 of this chapter on column supported embankments, FHWA NHI-04-001, Ground Improvement Methods (Elias et al., 2004) provides detailed information on prefabricated vertical drains, column supported embankments, and lightweight fill technologies.

7.2 APPLICATIONS

Reinforced embankments over weak foundations typically fall into one of two situations - construction over uniform deposits, and construction over local anomalies (Bonaparte, Holtz, and Giroud, 1985). The more common application is embankments, dikes, or levees constructed over very soft, saturated silt, clay, or peat layers (Figure 7-1). In this situation, the reinforcement is usually placed with its strong direction perpendicular to the centerline of the embankment, and plane strain conditions are assumed to prevail. Additional reinforcement with its strong direction oriented parallel to the centerline may also be required at the ends of the embankment.

The second reinforced embankment situation includes foundations below the embankment that are locally weak or contain voids. These zones or voids may be caused by sinkholes, thawing ice (thermokarsts), old streambeds, or pockets of silt, clay, or peat (Figure 7-1). In this application, the role of the reinforcement is to bridge over the weak zones or voids, and tensile reinforcement may be required in more than one direction. Thus, the strong direction of the reinforcing must be placed in proper orientation with respect to the embankment centerline (Bonaparte and Christopher, 1987).

Geotextiles may also be used as separators for displacement-type embankment construction (Holtz, 1989) and as stabilization layers to allow for embankment construction (see Chapter 5). In this application, the geotextile does not provide any reinforcement but only acts as a separator to maintain the integrity of the embankment as it displaces the subgrade soils. In this case, geotextile design is based upon constructability and survivability, and a high elongation material may be selected. Prefabricated geocomposite drains may also be placed as a drainage layer at the base of the embankment to allow for pore pressure dissipation and consolidation as an alternate to using clean, free draining granular fill for the first lift.

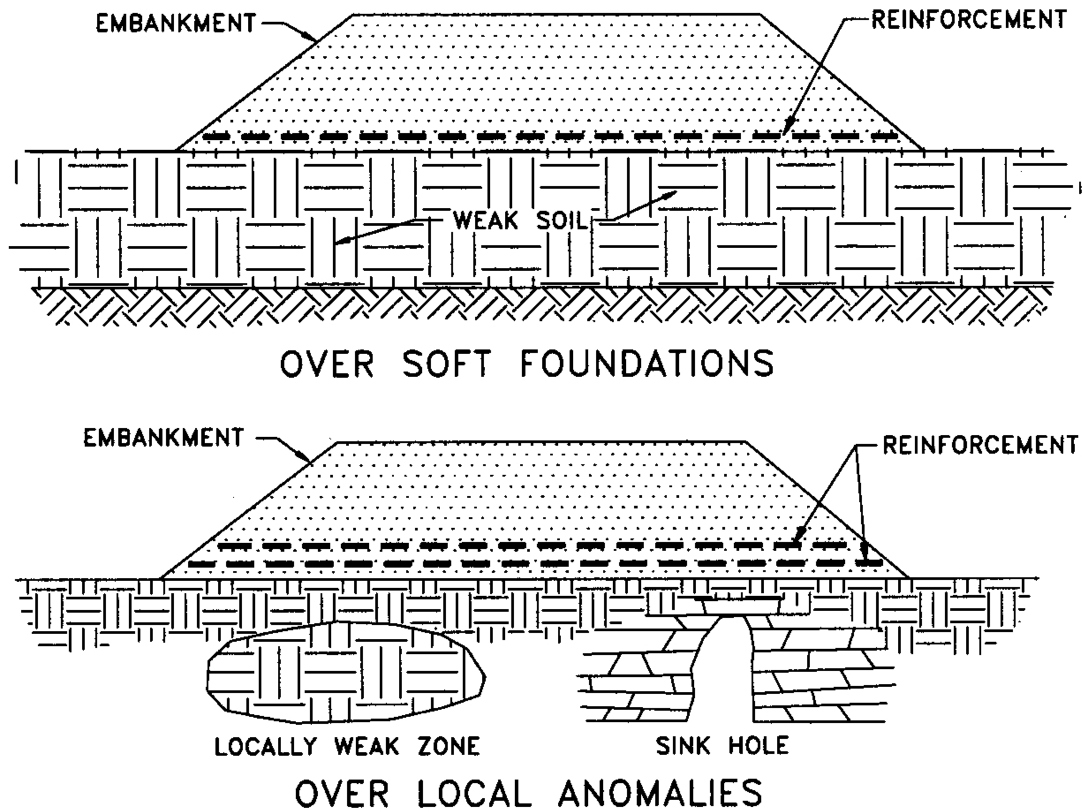


Figure 7-1. Reinforced embankment applications (after Bonaparte and Christopher, 1987).

7.3 DESIGN GUIDELINES FOR REINFORCED EMBANKMENTS ON SOFT SOILS

7.3-1 Design Considerations

As with ordinary embankments on soft soils, the basic design approach for reinforced embankments is to design against failure. The ways in which embankments constructed on soft foundations can fail have been described by Terzaghi and Peck (1967); Haliburton, Anglin and Lawmaster (1978 a and b); Fowler (1981); Christopher and Holtz (1985); and Koerner (1990), among others. Figure 7-2 shows unsatisfactory behavior that can occur in reinforced embankments. The three possible modes of failure indicate the types of stability analyses that are required. In addition, settlement of the embankment and potential creep of the reinforcement must be considered, although creep is only a factor if the creep rate in the reinforcement is greater than the strength gain occurring in the foundation due to consolidation. Because the most critical condition for embankment stability is at the end of construction, the reinforcement only has to function until the foundation soils gain sufficient strength to support the embankment.

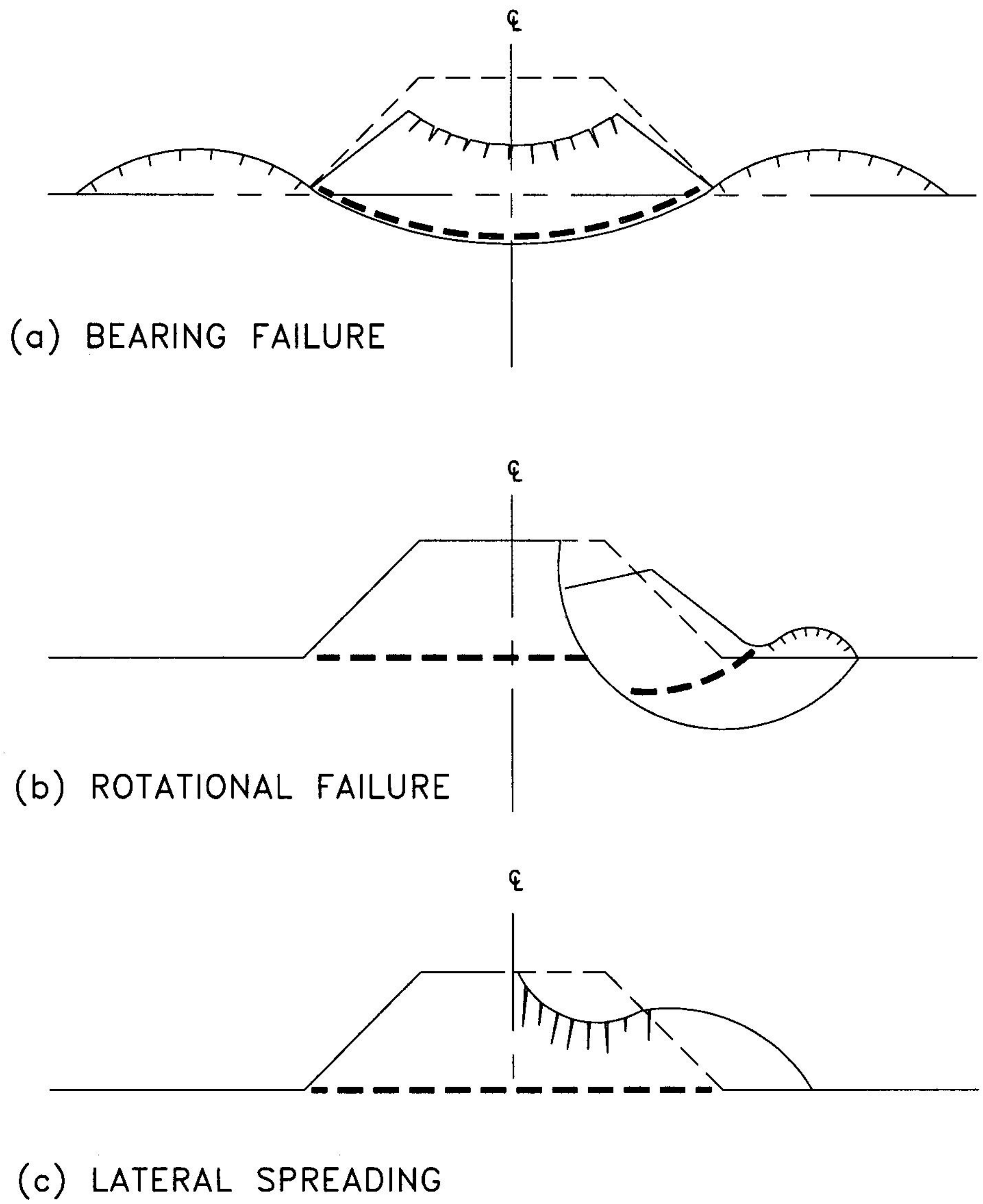


Figure 7-2. Reinforced embankments failure modes (after Haliburton et al., 1978).

The calculations required for stability and settlement utilize conventional geotechnical design procedures modified only for the presence of the reinforcement.

The stability of an embankment over soft soil is usually determined by the *total stress* method of analysis, which is conservative since the analysis generally assumes that no strength gain occurs in the compressible soil. The stability analyses presented in this text uses the *total stress* approach, because it is simple and appropriate for reinforcement design (Holtz, 1989).

It is always possible to calculate stability in terms of the effective stresses using the *effective stress* shear strength parameters. However, this calculation requires an accurate estimate of the field pore pressures to be made during the project design phase. Additionally, high-quality, undisturbed samples of the foundation soils must be obtained and K_o consolidated-undrained triaxial tests conducted in order to obtain the required design soil parameters. Because the prediction of in-situ pore pressures in advance of construction is not easy, it is essential that field pore pressure measurements using high quality piezometers be made during construction to control the rate of embankment filling. Preloading and staged embankment construction are discussed in detail by Ladd (1991). Note that by taking into account the strength gain that occurs with controlled rate (e.g. staged) embankment construction, lower strength and therefore lower cost reinforcement can be utilized. However; the time required for construction may be significantly increased and the costs of the site investigation, laboratory testing, design analyses, field instrumentation, and inspection are also greater.

The *total stress* design steps and methodology are detailed in the following section.

[The subjects of site investigation and laboratory testing, soil shear strength determination, and field instrumentation are addressed in detail in the following FHWA references: NHI-01-031 Subsurface Investigations – Geotechnical Site Characterization (NHI course No. 132031 reference manual); IF-02-034 Geotechnical Engineering Circular No. 5 Evaluation of Soil and Rock Properties; NHI-06-088 Soils and Foundations Workshop (NHI course No. 132012 reference manual); and HI-98-034 Geotechnical Instrumentation (NHI course No. 132041 reference manual).]

7.3-2 Design Steps

The following is a step-by-step procedure for design of reinforced embankments. Additional comments on each step can be found in Section 7.3-3.

STEP 1. Define embankment dimensions and loading conditions.

A. Embankment height, H

B. Embankment length

C. Width of crest

D. Side slopes, b/H

E. External loads
   1. surcharges
   2. temporary (traffic) loads
   3. dynamic loads

F. Environmental considerations
   1. frost action
   2. shrinkage and swelling
   3. drainage, erosion, and scour

G. Embankment construction rate
   1. project constraints
   2. anticipated or planned rate of construction

STEP 2. Establish the soil profile and determine the engineering properties of the foundation soil.

A. From a subsurface soils investigation, determine
   1. subsurface stratigraphy and soil profile
   2. groundwater table (location, fluctuation)

B. Engineering properties of the subsoils
   1. Undrained shear strength, c_u, for end of construction
   2. Drained shear strength parameters, c' and ϕ', for long-term conditions
   3. Consolidation parameters (C_c, C_r, c_v, σ_p')
   4. Chemical and biological factors that may be detrimental to the reinforcement

C. Variation of properties with depth and areal extent

STEP 3. Obtain engineering properties of embankment fill materials.

A. Classification properties

B. Moisture-density relationships

C. Shear strength properties

D. Chemical and biological factors that may be detrimental to the reinforcement

STEP 4. Establish minimum appropriate factors of safety and operational settlement criteria for the embankment. Suggested minimum factors of safety are as follows.

A. Bearing capacity:
Overall bearing capacity: 2.0
Local bearing capacity (i.e., lateral squeeze type failure): 1.3 to 2.0

B. Global (rotational) shear stability at the end of construction: 1.3

C. Internal shear stability, long-term: 1.5

D. Lateral spreading (sliding): 1.5

E. Dynamic loading: 1.1

F. Settlement criteria: dependent upon project requirements

STEP 5. Check bearing capacity.

A. When the thickness of the soft soil is much greater than the width of the embankment, use classical bearing capacity theory:

$$q_{ult} = \gamma_{fill}\, H = c_u\, N_c \qquad [7\text{-}1]$$

where N_c, the bearing capacity factor, is usually taken as 5.14 -- the value for a strip footing on a cohesive soil of constant undrained shear strength, c_u, with depth. This approach may underestimate the bearing capacity of reinforced embankments, as discussed in Section 7.3-3.

B. When the soft soil is of limited depth, perform a *lateral squeeze* analysis (Section 7.3-3).

STEP 6. Check rotational shear stability.

Perform a rotational slip surface analysis on the unreinforced embankment and foundation to determine the critical failure surface and the factor of safety against local shear instability.

A. If the calculated factor of safety is greater than the minimum required, then reinforcement is not needed. Check lateral embankment spreading (Step 7).

B. If the factor of safety is less than the required minimum, then calculate the required reinforcement strength, T_g, to provide an adequate factor of safety using Figure 7-3 or alternative solutions (Section 7.3-3), where:

$$T_g = \frac{FS(M_D) - M_R}{R\cos(\theta - \beta)}$$

STEP 7. Check lateral spreading (sliding) stability.

Perform a lateral spreading or sliding wedge stability analysis (Figure 7-4).

$$\boldsymbol{FS = \frac{F_{resisting}}{F_{driving}} = \frac{\frac{1}{2} H\, b\, \gamma \tan\phi_f}{\frac{1}{2} K_a\, \gamma\, H^2} = \frac{b \tan\phi_f}{K_a\, H}}$$

A. If the calculated factor of safety is greater than the minimum required, then reinforcement is not needed for this failure mode possibility.

B. If the factor of safety is inadequate, then determine the lateral spreading strength of reinforcement, T_{ls}, required -- see Figure 7-4b. Soil/geosynthetic cohesion, C_a, should be should be based on undrained direct shear tests on the soil/geosynthetic interface and assumed equal to 0 for extremely soft soils and low embankments. A cohesion value should be included with placement of the second and subsequent fills in staged embankment construction.

$$\boldsymbol{FS = \frac{2(b\, c_a + T_{ls})}{K_a\, \gamma\, H^2}}$$

where:

b	=	length of embankment side slope
H	=	height of embankment
K_a	=	coefficient of lateral earth pressure for embankment fill soil
ϕ'	=	friction angle of embankment soil
γ	=	unit weight of embankment soil
ϕ_{sg}	=	embankment soil to geosynthetic interface friction angle
c_u	=	cohesion (total stress) of foundation soil
c_a	=	adhesion of foundation soil to geosynthetic reinforcement

In absence of test data, the value of tan ϕ_{sg} may conservatively be taken as 2/3 tan ϕ'. In absence of test data, the value of c_a should be assumed to be 0.

C. Check sliding above the reinforcement. See Figure 7-4a.

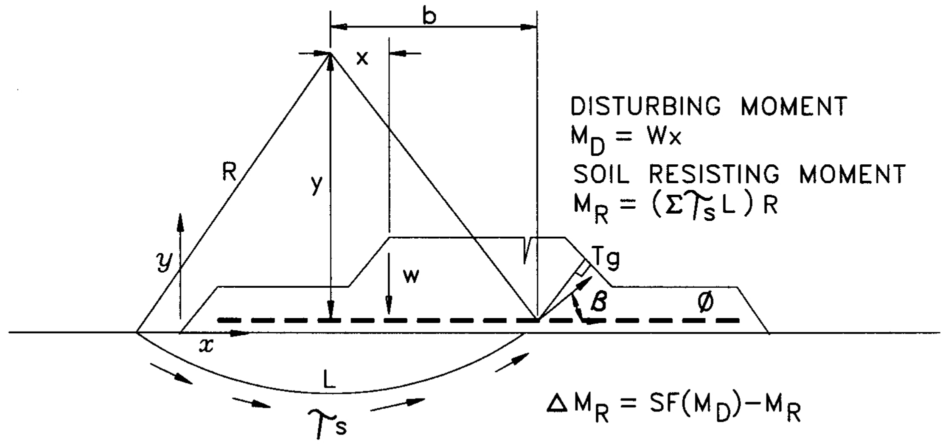
$$FS = \frac{b \tan \phi_{sg}}{K_a H}$$

STEP 8. Establish tolerable geosynthetic deformation requirements and calculate the required reinforcement modulus, J, based on wide width (ASTM D 4595) tensile testing.

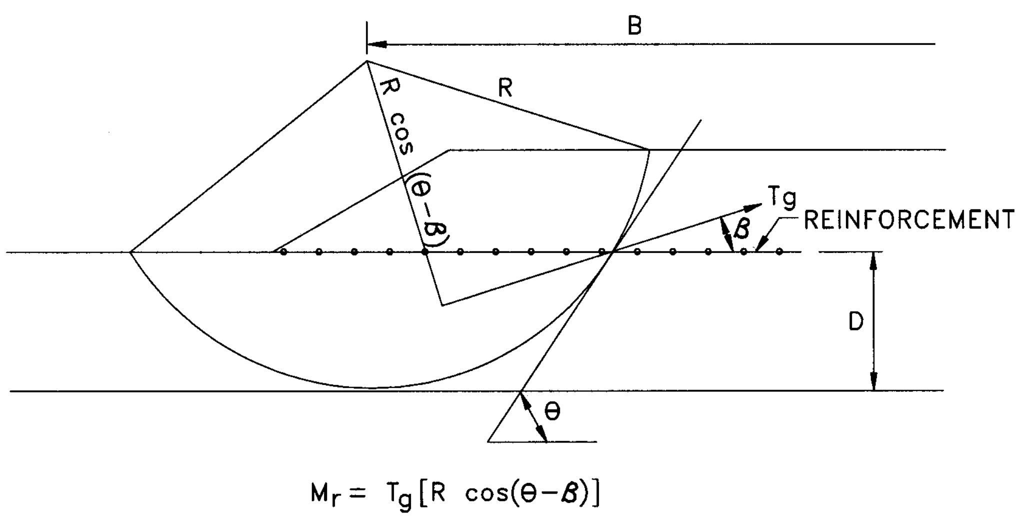
Reinforcement Modulus: $J = T_{ls} / \varepsilon_{geosynthetic}$ [7-2]

Recommendations for strain limits, based on type of fill soil materials and for construction over peats, are:

Cohesionless soils:	$\varepsilon_{geosynthetic}$ = 5 to 10%	[7-3]
Cohesive soils:	$\varepsilon_{geosynthetic}$ = 2%	[7-4]
Peats:	$\varepsilon_{geosynthetic}$ = 2 to 10%	[7-5]



(a) ROTATIONAL FAILURE MODEL



(b) ROTATIONAL FAILURE MODEL

Figure 7-3. Reinforcement required to provide rotational stability (a) Christopher and Holtz (1985) after Wager (1981); (b) Bonaparte and Christopher (1987) for the case in which the reinforcement does not increase soil strength.

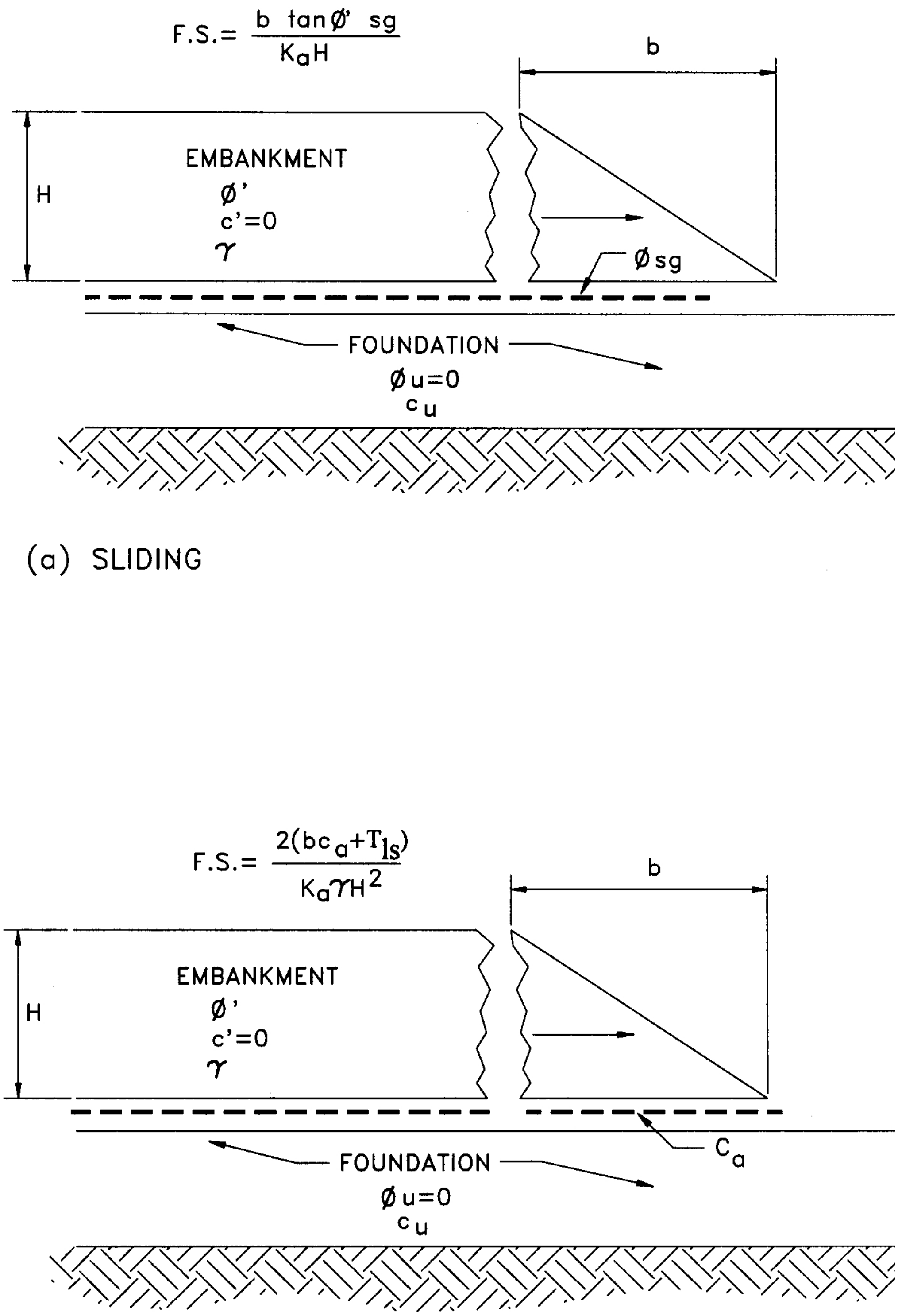


Figure 7-4. Reinforcement required to limit lateral embankment spreading (a) embankment sliding on reinforcement; (b) rupture of reinforcement and embankment sliding on foundation soil (from Bonaparte and Christopher, 1987).

STEP 9. Establish geosynthetic strength requirements in the embankment's longitudinal direction (i.e., direction of the embankment alignment).

A. Check bearing capacity and rotational slope stability at the ends of the embankment (Steps 5 and 6).

B. Use strength and elongation determined from Steps 7 and 8 to control embankment spreading during construction and to control bending following construction.

C. As the strength of the seams transverse to the embankment alignment control strength requirements, seam strength requirements are the higher of the strengths determined from Steps 9.A or 9.B.

STEP 10. Establish geosynthetic properties (Section 7.4).

A. Design strengths and modulus are based on the ASTM D 4595 wide width tensile test. This test standard permits definition of tensile modulus in terms of: (i) initial tensile modulus; (ii) offset tensile modulus; or (iii) secant tensile modulus. Furthermore, the secant modulus may be defined between any two strain points. Geosynthetic modulus for design of embankments should be determined using a secant modulus, defined with the zero strain point and design strain limit (i.e., 2 to 10%) point.

B. Seam strength is quantified with ASTM D 4884 test method, and is equal to the strength required in the embankment's longitudinal direction.

C. Soil-geosynthetic friction, ϕ_{sg}, based on ASTM D 5321 with on-site soils. For preliminary estimates, assume $\phi_{sg} = 2/3\phi$; for final design, testing is recommended.

D. Geotextile stiffness based on site conditions and experience. See Section 7.4-5.

E. Select survivability and constructability requirements for the geosynthetic based on site conditions, backfill materials, and equipment, using Tables 7-1, 7-2, and 7-3.

STEP 11. Estimate magnitude and rate of embankment settlement.

Use conventional geotechnical procedures and practices for this step.

STEP 12. Establish construction sequence and procedures.

See Section 7.8.

STEP 13. Establish construction observation requirements.

See Sections 7.8 and 7.9.

STEP 14. Hold preconstruction meetings.

Consider a *partnering* type contract with a disputes resolution board.

STEP 15. Observe construction and build with confidence (if the procedures outlined in these guidelines are followed!)

7.3-3 Comments on the Design Procedure

STEPS 1 and 2 need no further elaboration.

STEP 3. Obtain embankment fill properties.

Follow traditional geotechnical practice, except that the first few lifts of fill material just above the geosynthetic should be free-draining granular materials. This requirement provides the best frictional interaction between the geosynthetic and fill, as well as providing a drainage layer for excess pore water to dissipate from the underlying soils. Other fill materials may be used above this layer as long as the strain compatibility of the geosynthetic is evaluated with respect to the backfill materials (Step 8).

When a fill is placed on soft ground, the main driving force is from the weight of the embankment itself. It may be more advantageous to use a lightweight fill material so as to reduce the driving forces, thereby increasing the overall global stability of the fill. The reduction in driving force will depend upon the type of lightweight fill material used. The geotechnical properties of various types of lightweight fill materials are discussed in detail in FWHA NHI-04-001 Ground Improvement Methods Reference Manual (Elias, 2004). A secondary benefit of the use of lightweight fill material is the reduction in settlement under loading. The amount of settlement will be reduced proportionately to the reduction in load.

STEP 4. Establish design factors of safety.

The minimum factors of safety previously stated are recommended for projects with modern state-of-the-practice geotechnical site investigations and laboratory testing. Those factors may be adjusted depending on the method of analysis, type and use of facility being designed, the known conditions of the subsurface, the quality of the samples and soils testing, the cost of failure, the probability of unusual events occurring, and the engineer's previous experience on similar projects and sites. In short, all of the uncertainties in loads, analyses, and soil properties influence the choice of appropriate factors of safety. Typical factors of safety for unreinforced embankments also seem to be appropriate for reinforced embankments.

When the calculated factor of safety is greater than 1 but less than the minimum allowable factor of safety for design, say 1.3 or 1.5, then the geosynthetic provides an additional factor of safety or a *second line of defense* against failure. On the other hand, when the calculated factor of safety for the unreinforced embankment is significantly less than 1, the geosynthetic reinforcement is the difference between success and failure. In this latter case, construction considerations (Section 7.8) become crucial to the project success.

Maximum tolerable post-construction settlement and embankment deformations, which depend on project requirements, must also be established.

STEP 5. Check overall bearing capacity.

<u>Overall Bearing</u>

Reinforcement does not increase the overall bearing capacity of the foundation soil. If the foundation soil cannot support the weight of the embankment, then the embankment cannot be built. Thus, the overall bearing capacity of the entire embankment must be satisfactory before considering any possible reinforcement. As such, the vertical stress due to the embankment can be treated as an average stress over the entire width of the embankment, similar to a semi-rigid mat foundation.

The bearing capacity can be calculated using classical soil mechanics methods (Terzaghi and Peck, 1967; Vesic, 1975; Perloff and Baron, 1976; and U.S. Navy, 1982), which use limiting equilibrium-type analyses for strip footings, assuming logarithmic spiral failure surfaces on an infinitely deep foundation. These analyses are not appropriate if the thickness of the underlying soft deposit is small compared to the width of the embankment. In this case, high lateral stresses in the confined soft stratum beneath the embankment could lead to a *lateral squeeze*-type failure. Use of reinforced soils slopes (Chapter 8) or of mechanically stabilized earth walls (Chapter 9) can lead to high lateral stresses in underlying soft foundation soils. See following discussion for guidance on assessing this failure mechanism.

In a review of 40 reinforced embankment case histories, Humphrey and Holtz (1986) and Humphrey (1987) found that in many cases, the failure height predicted by classical bearing capacity theory was significantly less than the actual constructed height, especially if high strength geotextiles and geogrids were used as the reinforcement. Figure 7-5 shows the embankment height versus average undrained shear strength of the foundation. Significantly, four embankments failed at heights 2 m greater than predicted by Equation 7-1 (line B in Figure 7-5). The two reinforced embankments that failed below line B were either on peat or under reinforced (Humphrey, 1987). It appears that in many cases, the reinforcement enhances the beneficial effect the following factors have on stability:

- limited thickness or increasing strength with depth of the soft foundation soils (Rowe and Soderman, 1987 a and b; Jewell, 1988);
- the dry crust (Humphrey and Holtz, 1989);
- flat embankment side slopes (*e.g.*, Humphrey and Holtz, 1987); or
- dissipation of excess pore pressures during construction.

If the factor of safety for bearing capacity is sufficient, then continue with the next step. If not, consider increasing the embankment's width, flattening the slopes, adding toe berms, or improving the foundation soils by using stage construction and drainage enhancement or other alternatives, such as relocating the alignment or placing the roadway on an elevated structure.

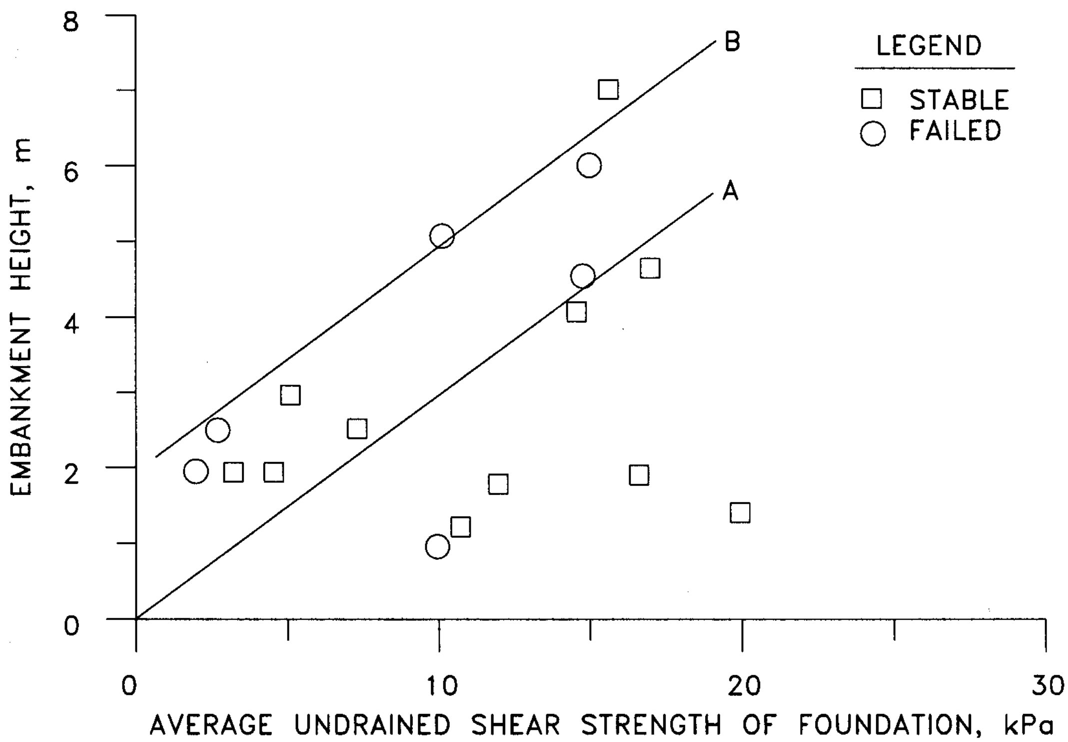


Figure 7-5. Embankment height versus undrained shear strength of foundation; line A: classical bearing capacity theory (Eq. 7-1); line B: line A + 2 m (after Humphrey, 1987).

Lateral Squeeze

High lateral stresses in a confined soft stratum beneath an embankment could lead to a *lateral squeeze*-type failure. Lateral squeeze-type failure of the foundation should be anticipated if γ_{fill} x $H_{fill} > 3c_u$, (see FHWA Soils and Foundation Manual, FHWA NHI-06-088, 2006) and a weak soil layer exists beneath the embankment to a depth that is less that the width of the embankment. The shear forces developed under the embankment should be compared to the corresponding shear strength of the soil. Approaches discussed by Jürgenson (1934), Silvestri (1983), and Bonaparte, Holtz and Giroud (1985), Rowe and Soderman (1987a), Hird and Jewell (1990), and Humphrey and Rowe (1991) are appropriate. The designer should be aware that the analysis for lateral squeeze is only approximate, and no single method is completely accepted by geotechnical engineers at present. When the depth of the soft layer, D_S, is greater than the base width of the embankment, general global bearing capacity and overall stability will govern the design.

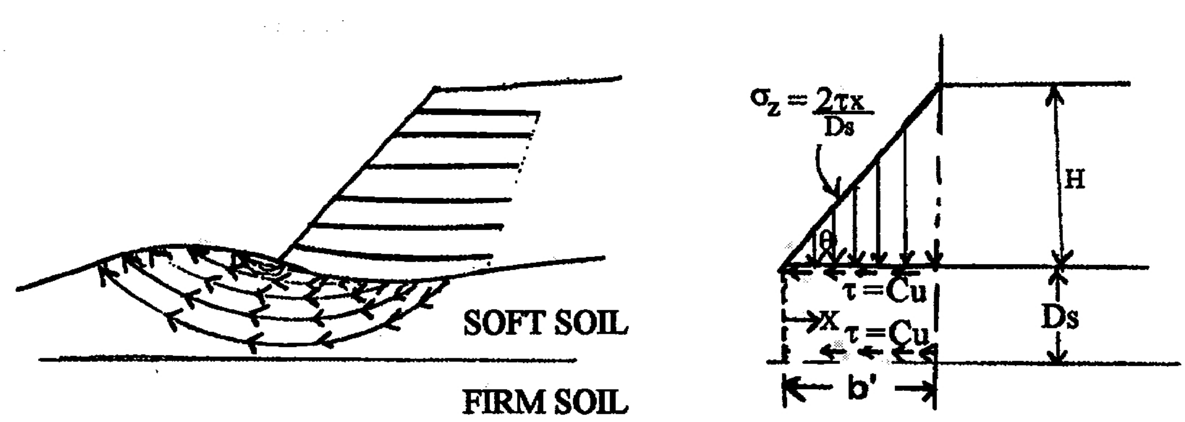
The approach by Silvestri (1983) is presented and demonstrated below for lateral squeeze failure at the toe of an embankment side slope. If a weak soil layer exists beneath the embankment to a limited depth D_S which is less than the width of the slope b' (see Figure 7-6), the factor of safety against failure by squeezing may be calculated from:

$$FS_{squeezing} = \frac{2\,c_u}{\gamma\, D_s\,(\tan\theta)} + \frac{4.14\,c_u}{H\,\gamma} \geq 1.3 \qquad [7\text{-}6]$$

where:

- θ = angle of slope.
- γ = unit weight of soil in slope.
- D_s = depth of soft soil beneath slope base of the embankment.
- H = height of slope.
- c_u = undrained shear strength of soft soil beneath slope.

Caution is advised and rigorous analysis (e.g, numerical modeling and/or extensive subsurface investigation with careful evaluation of c_u) should be performed when FS < 2. For factors of safety below 2, c_u should be confirmed through rigorous laboratory testing on undisturbed samples direct simple shear, evaluation of over consolidation ratio (e.g. Ladd, 1991), or triaxial compression with pore pressure measurements and/or field vane shear tests. Careful monitoring during construction will be required with piezometers, surface survey monuments (both within and outside the toe of the embankment), and inclinometers installed for construction control.



$$\boldsymbol{FS = \frac{2\,c_u}{\gamma\, D_s\,(\tan\theta)} + \frac{4.14\,c_u}{H\,\gamma}}$$

Figure 7-6. Local bearing failure (lateral squeeze).

If the foundation soils are cohesive and limited to a depth of less than the base width of the embankment, then local stability should be evaluated. As an example, assume that the foundation soils had an undrained shear strength of 16 kPa (320 psf) and extended to a depth of 3 m (10 ft) at which point the granular soils were encountered, and the embankment fill unit weight is 18.8 kN/m^3 (120 lb/ft^3). Constructing even a 4 m (13 ft) high embankment with a 2H:1V side slope would create a problem in accordance with equation 7-6 as follows.

$$FS_{squeezing} = \frac{2\,c_u}{\gamma\,D_s\,(\tan\theta)} + \frac{4.14\,c_u}{H\,\gamma} \Rightarrow$$

$$FS_{squeezing} = \frac{2\,(8\,kPa)}{(18.8\,kN/m^3)(3\,m)(\tan 26.6°)} + \frac{4.14\,(8\,kPa)}{4\,m\,(18.8\,kN/m^3)} = 1.01$$

Since $FS_{squeezing}$ is lower than the recommended 1.3, the stability conditions must be improved. This could be accomplished by either reducing the slope angle, use of lightweight embankment fill, or by placing a surcharge at the toe (which effectively reduces the slope angle). In addition, if the resulting factor of safety is less than 2, refinement of the analysis should be considered as previously discussed (i.e., careful evaluation of c_u, consider performing numerical modeling, and install instrumentation for construction control).

STEP 6. Check rotational shear stability.

The next step is to calculate the factor of safety against a circular failure through the embankment and foundation using classical limiting equilibrium-type stability analyses. If the factor of safety does not meet the minimum design requirements (Step 4), then the reinforcing tensile force required to increase the factor of safety to an acceptable level must be estimated.

This is done by assuming that the reinforcement acts as a stabilizing tensile force at its intersection with the slip surface being considered. The reinforcement thus provides the additional resisting moment required to obtain the minimum required factor of safety. The analysis is shown in Figure 7-3.

The analysis consists of determining the most critical failure surface(s) using conventional limiting equilibrium analysis methods. For each critical sliding surface, the driving moment (M_D) and soil resisting moment (M_R) are determined as shown in

Figure 7-3a. The additional resisting moment ΔM_R to provide the required factor of safety is calculated as shown in Figure 7-3b. Then one or more layers of geotextiles or geogrids with sufficient tensile strength at tolerable strains (Step 7) are added at the base of the embankment to provide the required additional resisting moment. If multiple layers are used, they must be separated by a granular layer and they must have compatible stress-strain properties (*e.g.*, the same type of reinforcement must be used for each layer).

A number of procedures have been proposed for determining the required additional reinforcement, and these are summarized by Christopher and Holtz (1985), Bonaparte and Christopher (1987), Holtz (1990), and Humphrey and Rowe (1991). The basic difference in the approaches is in the assumption of the reinforcement force orientation at the location of the critical slip surface (the angle ß in Figures 7-3a and 7-3b). It is conservative to assume that the reinforcing force acts horizontally at the location of the reinforcement (ß = 0). In this case, the additional reinforcing moment is equal to the required geosynthetic strength, T_g, times the vertical distance, y, from the plane of the reinforcement to the center of rotation, or:

$$\Delta M_R = T_g \, y \qquad [7\text{-}6a]$$

as determined for the most critical failure surface, shown in Figure 7-3a. This approach is conservative because it neglects any possible reinforcement reorientation along the alignment of the failure surface, as well as any confining effect of the reinforcement.

A less-conservative approach assumes that the reinforcement bends due to local displacements of the foundation soils at the onset of failure, with the maximum possible reorientation located tangent to the slip surface (ß = θ in Figure 7-3b). In this case,

$$\Delta M_R = T_g \, [R \cos (\theta - \beta)] \qquad [7\text{-}6b]$$

where,

θ = angle from horizontal to tangent line as shown in Figure 7-3.

Limited field evidence indicates that it is actually somewhere in between the horizontal and tangential (Bonaparte and Christopher, 1987) depending on the foundation soils, the depth of soft soil from the original ground line in relation to the width of the embankment (D/B ratio), and the stiffness of the reinforcement. Based on the minimal information available, the following suggestions are provided for selecting the orientation:

ß = 0 for brittle, strain-sensitive foundations soils (*e.g.*, leached marine clays) or where a crust layer is considered in the analysis for increased support;

ß = θ/2 for D/B < 0.4 and moderate to highly compressible soils (*e.g.*, soft clays, peats);

ß = θ for D/B. 0.4 highly compressible soils (*e.g.*, soft clays, peats); and reinforcement with high elongation potential (ϵ_{design} ≥ 10%), and large tolerable deformations; and

ß = 0 when in any doubt!

Other approaches, as discussed by Bonaparte and Christopher (1987), require a more rigorous analysis of the foundation soils deformation characteristics and the reinforcement strength compatibility.

In each method, the depth of the critical failure surface must be relatively shallow, *i.e.*, y in Figure 7-3a must be large, otherwise the geosynthetic contribution toward increasing the resisting moment will be small. On the other hand, Jewell (1988) notes that shallow slip surfaces tend to underestimate the driving force in the embankment, and both he and Leshchinsky (1987) have suggested methods to address this problem.

STEP 7. Check lateral spreading (sliding) stability.

A simplified analysis for calculating the reinforcement required to limit lateral embankment spreading is illustrated in Figure 7-4. For unreinforced as well as reinforced embankments, the driving forces result from the lateral earth pressures developed within the embankment and which must, for equilibrium, be transferred to the foundation by shearing stresses (Holtz, 1990). Instability occurs in the embankment when either:

1. the embankment slides on the reinforcement (Figure 7-4a); or
2. the reinforcement fails in tension and the embankment slides on the foundation soil (Figure 7-4b).

In the latter case, the shearing resistance of the foundation soils just below the embankment is insufficient to maintain equilibrium. Thus, in both cases, the reinforcement must have sufficient friction to resist sliding on the reinforcement plane, and the geosynthetic tensile strength must be sufficient to resist rupture as the potential sliding surface passes through the reinforcement.

The forces involved in the analysis of embankment spreading are shown in Figure 7-4 for the two cases above. The lateral earth pressures, usually assumed to be active, are of a maximum at the crest of the embankment. The factor of safety against embankment spreading is found from the ratio of the resisting forces to the actuating (driving) forces. The recommended factor of safety against sliding is 1.5 (Step 4). If the required soil-geosynthetic friction angle is greater than that reasonably achieved with the reinforcement, embankment soils and subgrade, then the embankment slopes must be flattened or berms must be added. Sliding resistance can be increased by the soil improvement techniques mentioned above. Generally, however, there is sufficient frictional resistance between geotextiles and geogrids commonly used for reinforcement and granular fill. If this is the case, then the resultant lateral earth pressures must be resisted by the tension in the reinforcement.

In the case where an MSE or RSS structure is founded at the end of the embankment (but not supporting a bridge structure) the length b may be taken as the reinforcement length, L, of the MSE or RSS structure. An MSE or RSS structure should only be included at the end of an embankment after the foundation soil has been adequately improved (i.e., through surcharging) to support such structures or other ground improvement techniques are employed, such as stabilization berms, lightweight fill, etc.

STEP 8. Establish tolerable deformation requirements for the geosynthetic.

Excessive deformation of the embankment and its reinforcement may limit its serviceability and impair its function, even if total collapse does not occur. Thus, an analysis to establish deformation limits of the reinforcement must be performed. The most common way to limit deformations is to limit the allowable strain in the geosynthetic. This is done because the geosynthetic tensile forces required to prevent failure by lateral spreading are not developed without some strain, and some lateral movement must be expected. Thus, geosynthetic modulus is used to control lateral spreading (Step 7). The distribution of strain in the geosynthetic is assumed to vary linearly from zero at the toe to a maximum value beneath the crest of the embankment. This is consistent with the development of lateral earth pressures beneath the slopes of the embankment.

For the assumed linear strain distribution, the maximum strain in the geosynthetic will be equal to twice the average strain in the embankment. Fowler and Haliburton (1980) and Fowler (1981) found that an average lateral spreading of 5% was reasonable, both from a construction and geosynthetic property standpoint. If 5% is the average strain, then the maximum expected strain would be 10%, and the geosynthetic modulus would be determined at 10% strain (Equation 7-3). However, it has been suggested that a modulus at 10% strain would be too large, and that smaller maximum values at, say 2 to 5%, are more appropriate.

If cohesive soils are used in the embankment, then the modulus should be determined at 2% strain to reduce the possibility of embankment cracking (Equation 7-4). Of course, if embankment cracking is not a concern, then these limiting reinforcement strain values could be increased. Keep in mind, however, that if cracking occurs, no resistance to sliding is provided. Further, the cracks could fill with water, which would add to the driving forces.

Additional discussion of geosynthetic deformation is given in Christopher and Holtz (1985 and 1989), Bonaparte, Holtz and Giroud (1985), Rowe and Mylleville (1989 and 1990), and Humphrey and Rowe (1991).

STEP 9. Establish geosynthetic strength requirements in the longitudinal direction.

Most embankments are relatively long but narrow in shape. Thus, during construction, stresses are imposed on the geosynthetic in the longitudinal direction, *i.e.*, in the along direction the centerline. Reinforcement may be also required for loadings that occur at bridge abutments, and due to differential settlements and embankment bending, especially over nonuniform foundation conditions and at the edges of soft soil deposit.

Because both sliding and rotational failures are possible, analyses procedures discussed in Steps 6 and 7 should be applied, but in the direction along the alignment of the embankment. This determines the longitudinal strength requirements of the geosynthetic. Because the usual placement of the geosynthetic is in strips perpendicular to the centerline, the longitudinal stability will be controlled by the strength of the transverse seams.

STEP 10. Establish geosynthetic properties.

See Section 7.4 for a determining the required properties of the geosynthetic.

STEP 11. Estimate magnitude and rate of embankment settlement.

Although not part of the stability analyses, both the magnitude and rate of settlement of the embankment should be considered in any reinforcement design. There is some evidence from finite element studies that differential settlements may be reduced somewhat by the presence of geosynthetic reinforcement. Long-term or consolidation settlements are not influenced by the geosynthetic, since compressibility of the foundation soils is not altered by the reinforcement, although the stress distribution may be somewhat different. Present recommendations provide for reinforcement design as outlined in Steps 6 - 10 above. Then use conventional geotechnical methods to estimate immediate, consolidation, and secondary settlements, as if the embankment was unreinforced (Christopher and Holtz, 1985).

Possible creep of reinforced embankments on soft foundations should be considered in terms of the geosynthetic creep rate versus the consolidation rate and strength gain of the foundation. If the foundation soil consolidates and gains strength at a rate faster than (or equal to) the rate the geosynthetic loses strength due to creep, there is no problem. Many soft soils such as peats, silts and clays with sand lenses have high permeability, therefore, they gain strength rapidly, but each case should be analyzed individually.

Time required for settlement can be substantially decreased with foundation drains. Consolidation of soft ground using vertical drains is a technique used since the 1920s. Today, the most common method is the use of *wick* drains, which can be best described as prefabricated vertical drains (PVDs), since drainage is via pressure, and not by wicking. PVDs are used to accelerate consolidation of soft saturated compressible soils under load. The most common use of PVDs is to accelerate consolidation for approach embankments at bridges or other embankment construction over soft soils, where the total post construction settlement is not acceptable.

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

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

The evaluation, design, cost, specification, and construction with PVDs are discussed in detail in FWHA NHI-04-001 Ground Improvement Methods Reference Manual (Elias, 2004). Filtration of the PVD geotextile should be evaluated following the guidelines in Chapter 2 of this manual.

STEP 12. Establish construction sequence and procedures.

The importance of proper construction procedures for geosynthetic reinforced embankments on very soft foundations cannot be over emphasized. A specific construction sequence is usually required to avoid failures during construction.

See Section 7.8 for details on site preparation, special construction equipment, geosynthetic placement procedures, seaming techniques, and fill placement and compaction procedures.

STEP 13. Establish construction observation requirements

See Sections 7.8 and 7.9.

A. Instrumentation. As a minimum, install piezometers, settlement points, and surface survey monuments. Also consider inclinometers to observe lateral movement with depth.

   Note that the purpose of the instrumentation in soft ground reinforcement projects is not for research but to verify design assumptions and to control and, usually, expedite construction.

B. Geosynthetic inspection. Be sure field personnel understand:
   - geosynthetic submittal for acceptance prior to installation;
   - testing requirements;
   - fill placement procedures; and
   - seam integrity verification.

STEP 14. Hold preconstruction meetings

It has been our experience that the more potential contractors know about the overall project, the site conditions, and the assumptions and expectations of the designers, the more realistically they can bid; and, the project is more successful. Prebid and preconstruction information meetings with contractors have been very successful in establishing a good, professional working relationship between owner, design engineer, and contractor. *Partnering* type contracts and a disputes resolution board can also be used to reduce problems, claims, and litigation.

STEP 15. Observe construction

Inspection should be performed by a trained and knowledgeable inspector, and good documentation of construction should be maintained.

7.4 SELECTION OF GEOSYNTHETIC AND FILL PROPERTIES

Once the design strength requirements have been established, the appropriate geosynthetic must be selected. In addition to its tensile and frictional properties, drainage requirements, construction conditions, and environmental factors must also be considered. Geosynthetic properties required for reinforcement applications are given in Table 7-1. The selection of appropriate fill materials is also an important aspect of the design. When possible, granular fill is preferred, especially for the first few lifts above the geosynthetic.

Table 7-1. Geosynthetic Properties Required For Reinforcement Applications.

Criteria and Parameter	Property[1]
Design Requirements:	
a. Mechanical	
Tensile strength	Wide width strength
Tensile modulus	Wide width strength
Seam strength	Wide width strength
Tension creep	Tension creep
Soil-geosynthetic friction	Soil-geosynthetic friction angle
b. Hydraulic	
Piping resistance	Apparent opening size
Permeability	Permeability
Constructability Requirements:	
Tensile strength	Grab strength
Puncture resistance	Puncture resistance
Tear resistance	Trapezoidal tear
Longevity:	
UV stability (if exposed)	UV resistance
Soil compatibility (where required)	Chemical; Biological
NOTE: 1. See Table 1-3 for specific test procedures.	

7.4-1 Geotextile and Geogrid Strength Requirements

The most important mechanical properties are the tensile strength and modulus of the reinforcement, seam strength, soil-geosynthetic friction, and system creep resistance.

The tensile strength and modulus values should preferably be determined by an in-soil tensile test. From research by McGown, Andrawes, and Kabir (1982) and others, we know that in-soil properties of many geosynthetics are markedly different than those from tests conducted in air. However, in-soil tests are not yet routine nor standardized, and the test proposed test methods need additional work. The practical alternate is to conservatively use a wide strip tensile test (ASTM D 4595) as a measure of the in-soil strength. This point is discussed by Christopher and Holtz (1985) and Bonaparte, Holtz, and Giroud (1985). Traditional grab or narrow-strip tensile tests are not appropriate for obtaining design properties of reinforcing geosynthetics.

Therefore, strength and modulus are based on the ASTM D 4595 wide width tensile test. This test standard permits definition of tensile modulus in terms of: (i) initial tensile modulus; (ii) offset tensile modulus; or (iii) secant tensile modulus. Furthermore, the secant modulus may be defined between any two strain points. Geosynthetic modulus for design of embankments should be determined using a secant modulus, defined with the zero strain point and design strain limit (i.e., 2 to 10%) point.

The following minimum criteria for tensile strength of geosynthetics are recommended.

1. For ordinary cases, determine the design tensile strength T_d (the larger of T_g and T_{ls}) and the required secant modulus at 2 to 10% strain.
2. The ultimate tensile strength T_{ult} obviously must be greater that the design tensile strength, T_d. Note that T_g includes an inherent safety factor against overload and sudden failure that is equal to the rotational stability safety factor. The tensile strength requirements should be increased to account for installation damage, depending on the severity of the conditions.
3. The strain of the reinforcement at failure should be at least 1.5 times the secant modulus strain to avoid brittle failure. For exceptionally soft foundations where the reinforcement will be subjected to very large tensile stresses during construction, the geosynthetic must have either sufficient strength to support the embankment itself, or the reinforcement and the embankment must be allowed to deform. In this case, an elongation at rupture of up to 50% may be acceptable. In either case, high tensile strength geosynthetics and special construction procedures (Section 7.8) are required.
4. If there is a possibility of tension cracks forming in the embankment or high strain levels occurring during construction (such as might occur, for example,

with cohesive embankments), the lateral spreading strength, T_{ls}, at 2% strain should be required.

5. The required lateral spreading strength, T_{ls}, should be increased to account for creep and installation damage as the creep potential of the geosynthetic depends on the creep potential of the foundation. If significant creep is expected in the foundation, the creep potential of the geosynthetic at design stresses should be evaluated, recognizing that strength gains in the foundation will reduce the creep potential. Installation damage potential will depend on the severity of the conditions.
6. Strength requirements must be evaluated and specified for both the machine and cross machine directions of the geosynthetic. Usually, the seam strength controls the cross machine geosynthetic strength requirements.

Depending on the strength requirements, geosynthetic availability, and seam efficiency, more than one layer of reinforcement may be necessary to obtain the required tensile strength. If multiple layers are used, a granular layer of 200 to 300 mm must be placed between each successive geosynthetic layer or the layers must be mechanically connected (*e.g.*, sewn) together. Also, the geosynthetics must be strain compatible; that is, the same type of geosynthetic should be used for each layer.

For soil-geosynthetic friction values, either direct shear or pullout tests should be utilized. If test values are not available, Bell (1980) recommends that for sand embankments, the soil-geosynthetic friction is from $^2/_3\phi$ up to the full ϕ of the sand. Since these early recommendations, a number of direct shear and pullout tests have been performed on both geogrids and geotextiles and the recommendations still apply. It is recommended that in the absence of tests, a soil-geosynthetic friction angle of 2/3 ϕ should be conservatively used for granular fill placed directly own the geosynthetic. For clay soils, friction tests are definitely warranted and should be performed under all circumstances.

The creep properties of geosynthetics in reinforced soil systems are not well established. In-soil creep tests are possible but are far from routine today. For design, it is recommended that the working stress be kept much lower than the creep limit of the geosynthetic. Values of 40 to 60% of the ultimate stress are typically satisfactory for this purpose. A polyester will probably have less creep than a polypropylene or a polyethylene. Live loads versus dead loads also must be taken into account. Short-term live loadings are much less detrimental in terms of creep than sustained dead loads. And finally, as discussed in Section 7.3-3 Step 11, the relative rates of deformation of the geosynthetic versus the consolidation and strength gain of the foundation soil must be considered. In most cases, creep is not an issue in reinforced embankment stability.

7.4-2 Drainage Requirements

The geosynthetic must allow for free vertical drainage of the foundation soils to reduce pore pressure buildup below the embankment. Pertinent geosynthetic hydraulic properties are piping resistance and permeability (Table 7-1). It is recommended that the permeability of the geosynthetic be at least 10 times that of the underlying soil. Permeability values could be based on consolidation tests and taken at initial load levels to simulate initial placement of fill. The opening size should be selected based on the requirements of Section 2.3. The opening size should be a maximum to reduce the risk of clogging, while still providing retention of the underlying soil.

7.4-3 Environmental Considerations

For most embankment reinforcement situations, geosynthetics have a high resistance to chemical and biological attack; therefore, chemical and biological compatibility is usually not a concern. However, in unusual situations such as very low (*i.e.*, < 3) or very high *(i.e.*, > 9) pH soils, or other unusual chemical environments -- such as in industrial areas or near mine or other waste dumps -- the chemical compatibility of the polymer(s) in the geosynthetic should be checked to assure it will retain the design strength at least until the underlying subsoil is strong enough to support the structure without reinforcement.

7.4-4 Constructability (Survivability) Requirements

In addition to the design strength requirements, the geotextile or geogrid must also have sufficient strength to survive construction. If the geotextile is ripped, punctured, or torn during construction, support strength for the embankment structure will be reduced and failure could result. Constructability property requirements are listed in Table 7-1. (These are also called survivability requirements.) Tables 7-2 and 7-3 were developed by Haliburton, Lawmaster, and McGuffey (1982) specifically for reinforced embankment construction with varying subgrade conditions, construction equipment, and lift thicknesses (see also Christopher and Holtz, 1985). The specific property values are provided in Table 7-4 and Table 7-5. The high and moderate class conditions are taken directly from survivability tables in Chapter 5 for road construction (e.g., Table 5-3 and 5-4 from AASHTO M-288 Specification (2005) for geotextiles and Table 5-5 for geogrids) and are equivalent to Class 1 and Class 2 geosynthetics, respectively. The very high class requires greater strength than the requirements in Chapter 5 due to the possibility of constructing embankments on uncleared subgrade, which is a much harsher condition than anticipated for roads. For all critical applications, high to very high survivability geotextiles and geogrids are recommended. As the construction of the first lift of the embankment is analogous to construction of a temporary haul road, survivability requirements discussed in Section 5.9 are also appropriate here.

Table 7-2. Required Degree Of Geosynthetic Survivability as a Function of Subgrade Conditions And Construction Equipment.

SUBGRADE CONDITIONS	Construction Equipment and 150 to 300 mm Cover Material Initial Lift Thickness		
	Low Ground Pressure Equipment (30 kPa)	Medium Ground Pressure Equipment (> 30 kPa 60 kPa)	High Ground Pressure Equipment (>60 kPa)
Subgrade has been cleared of all obstacles except grass, weeds, leaves, and fine wood debris. Surface is smooth and level, and shallow depressions and humps do not exceed 150 mm in depth and height. All larger depressions are filled. Alternatively, a smooth working table may be placed.	Moderate/Low	Moderate	High
Subgrade has been cleared of obstacles larger than small- to moderate-sized tree limbs and rocks. Tree trunks and stumps should be removed or covered with a partial working table. Depressions and humps should not exceed 450 mm in depth and height. Larger depressions should be filled.	Moderate	High	Very High
Minimal site preparation is required. Trees may be felled, delimbed, and left in place. Stumps should be cut to project not more than 150 mm above subgrade. Geosynthetic may be draped directly over the tree trunks, stumps, large depressions and humps, holes, stream channels, and large boulders. Items should be removed only if, where placed, the Geosynthetic and cover material over them will distort the finished road surface.	High	Very High	Not Recommended

NOTES:
1. Recommendations are for 150 to 300 mm initial thickness. For other initial lift thickness:
   300 to 450 mm:Reduce survivability requirement one level
   450 to 600 mm:Reduce survivability requirement two levels
   > 600 mm:Reduce survivability requirement three levels
2. For special construction techniques such as prerutting, increase survivability requirement one level.
3. Placement of excessive initial cover material thickness may cause bearing failure of soft subgrades.
4. Note that equipment used for embankment construction (even *High Ground Pressure* equipment) have significantly lower ground contact pressures than equipment used for roadway construction (Table 5-2).

Table 7-3. Required Degree of Geosynthetic Survivability as a Function of Cover Material and Construction Equipment.

CONSTRUCTION		COVER MATERIAL		
		Fine sand to +50 mm diameter gravel, rounded to subangular	Coarse aggregate with diameter up to one-half proposed lift thickness, may be angular	Some to most aggregate with diameter greater than one-half proposed lift thickness, angular and sharp-edged, few fines
150 to 300 mm Initial Lift Thickness	Low ground pressure equipment (30 kPa)	Moderate/Low	Moderate	High
	Medium ground pressure equipment (>30 kPa, 60 kPa)	Moderate	High	Very High
300 to 450 mm Initial Lift Thickness	Medium ground pressure equipment (>30 kPa, 60 kPa)	Moderate/Low	Moderate	High
	High ground pressure equipment (>60 kPa)	Moderate	High	Very High
450 to 600 mm Initial Lift Thickness	High ground pressure equipment (>60 kPa)	Moderate/Low	Moderate	High
> 600 mm Initial Lift Thickness	High ground pressure equipment (>60 kPa)	Moderate/Low	Moderate/Low	Moderate

NOTES:
1. For special construction techniques such as prerutting, increase geosynthetic survivability requirement one level.
2. Placement of excessive initial cover material thickness may cause bearing failure of soft subgrades.
3. Note that equipment used for embankment construction (even *High Ground Pressure* equipment) have significantly lower ground contact pressures than equipment used for roadway construction (Table 5-2).

Table 7-4. Minimum Geotextile Property Requirements[1,2,3] for Geotextile Survivability (after AASHTO, 1997)

Property	ASTM Test Method	Units	Required Degree of Geotextile Survivability		
			Very High	High	Moderate
Grab Strength	D 4632	N	(see Note 4)	1400	1100
Tear Strength	D 4533	N	(see Note 4)	500	400
Puncture Strength	D 6241	N	(see Note 4)	2750	2200

NOTES:
1. Acceptance of geotextile material shall be based on ASTM D 4759.
2. Acceptance shall be based upon testing of either conformance samples obtained using Procedure A of ASTM D 4354, or based on manufacturer's certifications and testing of quality assurance samples obtained using Procedure B of ASTM D 4354.
3. Minimum; use value in weaker principal direction. All numerical values represent minimum average roll value (*i.e.*, test results from any sampled roll in a lot shall meet or exceed the minimum values in the table). Lot samples according to ASTM D 4354.
4. Recommend survivability of candidate "Very High" survivability geotextile(s) be demonstrated on a field/project basis or the use of a "High" survivability geotextile as a sacrificial layer.

7.4-5 Stiffness and Workability

For extremely soft soil conditions, geosynthetic stiffness or workability may be an important consideration. The *workability* of a geosynthetic is its ability to support workpersons during initial placement and sewing operations and to support construction equipment during the first lift placement. Workability is generally related to geosynthetic stiffness; however, stiffness evaluation techniques and correlations with field workability are very poor (Tan, 1990). In the absence of any other stiffness information, ASTM Standard D 1388, Option A using a 50 x 300 mm or longer specimen is recommended (see Christopher and Holtz, 1985). The values obtained should be compared with actual field performance to establish future design criteria. The workability guidelines based on subgrade CBR (Christopher and Holtz, 1985) are satisfactory for CBR > 1.0. For very soft subgrades, much stiffer geosynthetics are required. Other aspects of field workability such as water absorption and bulk density, should also be considered, especially on very soft sites.

Table 5-5. Geogrid Survivability Property Requirements[1,2,3]

Property	Test Method	Units	Requirement	
SURVIVABILITY			Geogrid Class[4]	
			CLASS 1[5]	CLASS 2
Ultimate Multi-Rib Tensile Strength	ASTM D 6637	kN/m	19	12
Junction Strength[6]	GSI GRI GG2	N	110[5]	110
Ultraviolet Stability (Retained Strength)	ASTM D 4355	%	50% after 500 hours of exposure	
OPENING CHARACTERISTICS				
Opening Size	Direct measure	mm	12.5 to 75 mm and Opening Size > D_{50} of aggregate above geogrid	
Separation	ASTM D 422	mm	D_{85} of aggregate above geogrid < 5 D_{85} subgrade Other wise use separation geotextile with geogrid	

NOTES:
1. Acceptance of geogrid material shall be based on ASTM D 4759.
2. Acceptance shall be based upon testing of either conformance samples obtained using Procedure A of ASTM D 4354, or based on manufacturer's certifications and testing of quality assurance samples obtained using Procedure B of ASTM D 4354.
3. Minimum; use value in weaker principal direction. All numerical values represent minimum average roll value (*i.e.*, test results from any sampled roll in a lot shall meet or exceed the minimum values in the table). Lot samples according to ASTM D 4354.
4. Class 1 is considered a "High" survivability geogrid and Class 2 as a "Moderate" survivability geogrid. Recommend survivability of candidate "Very High" survivability geogrid(s) be demonstrated on a field/project basis or the use of a "High" survivability geogrid as a sacrificial layer in conditions requiring "Very High" survivability.
5. Default geogrid selection. The engineer may specify a Class 2 geogrid for moderate survivability conditions, see Table 5-2.
6. Junction strength requirements have not been fully supported by data, and until such data is established, manufacturers shall submit data from full scale installation damage tests in accordance with ASTM D 5818 documenting integrity of junctions. For soft soil applications, a minimum of 150 mm of cover aggregate shale be placed over the geogrid and a loaded dump truck used to traverse the section a minimum number of passes to achieve 4 inches of rutting. A photographic record of the geogrid after exhumation shall be provided, which clearly shows that junctions have not been displaced or otherwise damaged during the installation process.

7.4-6 Fill Considerations

The first lift of fill material just above the geosynthetic should be free- draining granular materials. This requirement provides the best frictional interaction between the geosynthetic and fill, as well as a drainage layer for excess pore water dissipation of the underlying soils. Other lower permeability, (preferably granular) fill materials may be used above this layer as long as the strain compatibility of the geosynthetic is evaluated with respect to the backfill material, as discussed in Section 7.3-3, Step 8.

Most reinforcement analyses assume that the fill material is granular. In fact, in the past the use of cohesive soils together with geosynthetic reinforcement has been discouraged. This may be an unrealistic restriction, although there are problems with placing and compacting cohesive earth fills on especially soft subsoils. Furthermore, the frictional resistance between geosynthetics and cohesive soils is problematic. It may be possible to use composite embankments. Cohesionless fill could be used for the first 0.5 to 1 m; then the rest of the embankment could be constructed to grade with locally available materials.

7.5 DESIGN EXAMPLE

DEFINITION OF DESIGN EXAMPLE

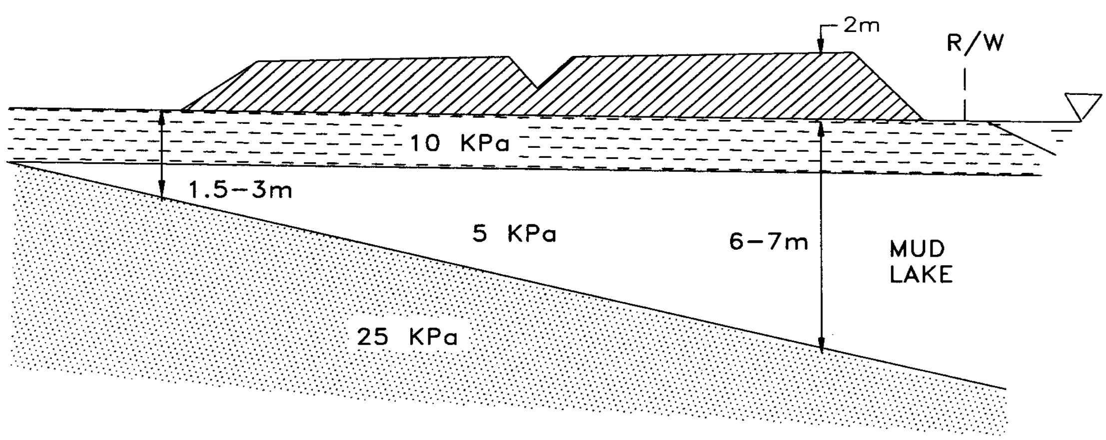
- Project Description: A 4-lane highway is to be constructed over a peat bog. Alignment and anticipated settlement require construction of an embankment with an average height of 2 m. See project cross section figure.
- Type of Structure: embankment supporting a permanent paved road
- Type of Application: geosynthetic reinforcement
- Alternatives:
  i) excavate and replace - wetlands do not allow;
  ii) lightweight fill - high cost;
  iii) stone columns - soils too soft;
  iv) drainage and surcharge - yes; or
  v) very flat (8H:1V) slope - right-of-way restriction

GIVEN DATA

- Geometry - as shown in project cross section figure
- Geosynthetic - geotextile (a geogrid also may be used for this example problem; however, this example represents an actual case history where a geotextile was used)
- Soils - subsurface exploration indicates c_u = 5 kPa in weakest areas

- soft soils are underlain by firmer soils of $c_u = 25$ kPa
- embankment fill soil will be sands and gravel
- lightweight fill costs $250,000 more than sand/gravel

- Stability
  - Stability analyses of the unreinforced embankment were conducted with the STABL computer program. The most critical condition for embankments on soft soils is end-of-construction case; therefore, UU (unconsolidated, undrained) soil shear strength values are used in analyses.
  - Results of the analyses:
    a. With 4:1 side slopes and sand/gravel fill (= 21.7 kN/m^3), FS ~ 0.72.
    b. Since FS was substantially less than 1 for 4H:1V slopes, flatter slopes were evaluated, even though additional right-of-way would be required. With 8:1 side slopes and sand/gravel fill (= 21.7 kN/m^3), a FS ~ 0.87 was computed.
    c. Light-weight fill (= 15.7 kN/m^3) was also considered, with it, the FS varied between ~ 0.90 to 1.15
  - Transportation Department required safety factors are:
    Fs_{min} > 1.5 for *long-term* conditions
    FS_{allow} ~ 1.3 for *short-term* conditions



Project Cross Section

REQUIRED

Design geotextile reinforcement to provide a stable embankment.

DEFINE

A. Geotextile function(s):
B. Geotextile properties required:
C. Geotextile specification:

SOLUTION

A. Geotextile function(s):

Primary	-	reinforcement (for short-term conditions)
Secondary	-	separation and filtration

B. Geotextile properties required:

tensile characteristics
interface shear strength
survivability
apparent opening size (AOS)

DESIGN

Design embankment with geotextile reinforcement to meet short-term stability requirements.

STEP 1. DEFINE DIMENSIONS AND LOADING CONDITIONS

See project cross section figure.

STEP 2. SUBSURFACE CONDITIONS AND PROPERTIES

Undrained shear strength provided in given data. Design for end-of-construction. Long-term design with drained shear strength parameters not covered within this example.

STEP 3. EMBANKMENT FILL PROPERTIES

sand and gravel, with
$\gamma_m = 21.7\ kN/m^3$ $\phi' = 35°$

STEP 4. ESTABLISH DESIGN REQUIREMENTS

- Transportation Department required safety factors are:
  $FS_{min} > 1.5$ for *long-term* conditions
  FS_{min} ˜ 1.3 for *short-term* conditions

- settlement
  Primary consolidation must be completed prior to paving roadway.
  A total fill height of 2 m is anticipated to reach design elevation. **This height includes the additional fill material thickness to compensate for anticipated settlements.**

STEP 5. CHECK OVERALL BEARING CAPACITY

Recommended minimum safety factor (section 7.3-2) is 2.

A. Overall bearing capacity of soil, ignoring *footing* size is

$q_{ult} = c\ N_c$

$q_{ult} = 5\text{ kPa } x\ 5.14 = 25.7\text{ kPa}$

Considering depth of embedment (*i.e.*, shearing will have to occur through the embankment for a bearing capacity failure) the bearing capacity is more accurately computed (see Meyerhof) as follows.

$N_c = 4.14 + 0.5\ B/D$ where, B = the base width of the embankment ˜(31 m), and
D = the average depth of the soft soil ˜(4.5 m)

$N_c = 4.14 + 0.5\ (31\text{ m} / 4.5\text{ m}) = 7.6$

$q_{ult} = 5\text{ kPa } x\ 7.6 = 38\text{ kPa}$

maximum load, $P_{max} = {}_{.\ m} H$

w/o a geotextile -

$P_{max} = 21.7\text{ kN/m}^3\ x\ 2\text{ m} = 43.4\text{ kPa}$

implies FS = 38 / 43.4 = 0.88 NO GOOD

with a geotextile, and assuming that the geotextile will result in an even distribution of the embankment load over the width of the geotextile (*i.e.*, account for the slopes at the embankment edges),

$P_{avg} = A_{E\ m} / B$ where, A = cross section area of embankment, and
B = base width of the embankment

$P_{avg} = \{[½\ (31\text{ m} + 15\text{ m})\ 2\text{ m}]\ 21.7\text{ kN/m}^3\} / 31\text{ m}$

$P_{avg} = 32.2\text{ kPa} < q_{ult}$ worst case Safety Factor Marginal

Add berms to increase bearing capacity. Berms, 3 m wide, can be added within the existing right-of-way, increasing the base width to 37 m. With this increase in width,

$N_c = 4.14 + 0.5\ (37\text{ m} / 4.5\text{ m}) = 8.3$

$q_{ult} = 5\text{ kPa x } 8.3 = 41.5\text{ kPa}$

and,

$P_{avg} = 32.2\text{ kPa}\ (31 / 37) = 27.0\text{ kPa}$

FS = 41.5 kPa / 27.0 kPa = 1.54 Safety Factor O.K.

B. Lateral squeeze

From FHWA Foundation Manual (Cheney and Chassie, 1993) -

If$_{.\ fill}$ x H_{fill} > 3c, then lateral squeeze of the foundation soil can occur. Since P_{max} = 43.4 kPa is much greater than 3c, even considering the crust layer (c = 10 kPa), a rigorous lateral squeeze analysis was performed using the method by Jürgeson (1934). In this method, the lateral stress beneath the toe of the embankment is determined through charts or finite element analysis and compared to the shear strength of the soil. This method indicated a safety factor of approximately 1 for the 31 m base width. Adding the berm and extending the reinforcement to the toe of the berm decreases the potential for lateral squeeze as the lateral stress is reduced at the toe of the berm. The berms increased $FS_{SQUEEZZE}$ to greater than 1.5.

Also, comparing the reinforced design with Figure 7-5 indicates that the reinforced structure should be stable.

STEP 6. PERFORM ROTATIONAL SHEAR STABILITY ANALYSIS

Recommended minimum safety factor at end of construction (section 7.3-2) is 1.3.

The critical unreinforced failure surface is found through rotational stability methods. For this project, STABL4M was used and the critical, unreinforced surface FS = 0.72. As the soil supporting the embankment was highly compressible peat, the reinforcement was assumed to rotate such that β = θ (Figure 7-3 and Eq. 7-4b). Thus,

$$FS_{req} = \frac{M_R + T_g R}{M_D} \geq 1.3$$

$$T_g = \frac{1.3 M_D - M_R}{R}$$

therefore, $\tilde{T}_g \approx 263$ kN/m

Feasible - yes. Geosynthetics are available which exceed this strength requirement, especially if multiple layers are used. For this project, an installation damage factor of approximately equal to 1.0, and 2 layers were used:

Bottom: 90 kN/m

Top: 180 kN/m

The use of 2 layers allowed the lower cost bottom material to be used over the full embankment plus berm width, while the higher strength and more expensive geotextile was only placed under the embankment section where it was required.

STEP 7. CHECK LATERAL SPREADING (SLIDING) STABILITY

Recommended minimum safety factor (section 7.3-2) is 1.5.

A. from Figure 7-4b:

$T = FS \, x \, P_A = FS \, x \, 0.5 \, K_a \, \gamma_m \, H^2$

$T = 1.5 \, (0.5) \, [\tan^2 (45 - 35/2)] \, (21.7 \text{ kN/m}^3) \, (2 \text{ m})^2$

$T = 17.6$ kN/m

Use Reduction Factors (RF) = 3 for creep and 1 installation damage

therefore, $T_{ls} = 53$ kN/m

$T_{ls} < T_g$, therefore $T_{design} = T_g = 263$ kN/m

B. check sliding:

$$FS = \frac{b \tan \phi_{sg}}{K_a H}$$

$$FS = \frac{8m \times \tan 23}{0.27 \times 2m}$$

FS > 6, OK

STEP 8. ESTABLISH TOLERABLE DEFORMATION (LIMIT STRAIN) REQUIREMENTS

For cohesionless sand and gravel over deformable peat use. = 10%

STEP 9. EVALUATE GEOSYNTHETIC STRENGTH REQUIRED IN LONGITUDINAL DIRECTION

From Step 7,

use $T_L = T_{ls} =$ 53 kN/m for reinforcement and seams in the cross machine (X-MD) direction

STEP 10. ESTABLISH GEOSYNTHETIC PROPERTIES

A. Design strength and elongation based upon ASTM D 4595

Ultimate tensile strength

$T_{d1} = T_{ult} \geq$ 90 kN/m in MD - Layer 1

$T_{d2} = T_{ult} \geq$ 180 kN/m in MD - Layer 2

$T_{ult} \geq$ 53 kN/m in X-MD - both layers

Reinforcement Modulus, J

$J = T_{ls} / 0.10 =$ 530 kN/m for limit strain of 10%

$J \geq$ 530 kN/m - MD and X-MD, both directions

B. seam strength

$T_{seam} \geq$ 53 kN/m with controlled fill placement

C. soil-geosynthetic adhesion

from testing, per ASTM D 5321, $\phi_{sg} \geq 23$

D. geotextile stiffness based upon site conditions and experience

E. survivability and constructability requirements

Assume: 1. medium ground pressure equipment

2. 300 mm first lift

3. uncleared subgrade

Use a *Very High Survivability* geotextile (from Tables 7-2 and 7-3). Therefore, from Table 7-4, the survivability of candidate geotextile reinforcements shall be demonstrated on a field/project basis or a "High" survivability geotextile, meeting the minimum average roll values listed below, may be used as a sacrificial layer.

Property	ASTM Test Method	Minimum Strength (N)
Grab Strength	D 4632	1400
Tear Resistance	D 4533	500
Puncture Strength	D 4833	2750

Drainage and filtration requirements -

Need grain size distribution of subgrade soils

Determine: maximum AOS for retention

minimum $k_g > k_s$

minimum AOS for clogging resistance

Complete Steps 11 through 15 to finish design.

STEP 11. PERFORM SETTLEMENT ANALYSIS

STEP 12. ESTABLISH CONSTRUCTION SEQUENCE REQUIREMENTS

STEP 13. ESTABLISH CONSTRUCTION OBSERVATION REQUIREMENTS

STEP 14. HOLD PRECONSTRUCTION MEETING

STEP 15. OBSERVE CONSTRUCTION

7.6 SPECIFICATIONS

Because the reinforcement requirements for soft-ground embankment construction will be project and site specific, standard specifications, which include suggested geosynthetic properties, are not appropriate, and special provisions or a separate project specification must be used. The following example includes most of the items that should be considered in a reinforced embankment project.

HIGH STRENGTH GEOTEXTILE FOR EMBANKMENT REINFORCEMENT

(from Washington Department of Transportation, October 27, 1997)

Description

This work shall consist of furnishing and placing construction geotextile in accordance with the details shown in the plans, these specifications, or as directed by the Engineer.

Materials

Geotextile and Thread for Sewing

The material shall be a woven geotextile consisting only of long chain polymeric filaments or yarns formed into a stable network such that the filaments or yarns retain their position relative to each other during handling, placement, and design service life. At least 95 percent by mass of the of the material shall be polyolefins or polyesters. The material shall be free from defects or tears. The geotextile shall be free of any treatment or coating which might adversely alter its hydraulic or physical properties after installation. The geotextile shall conform to the properties as indicated in Table 1.

Thread used shall be high strength polypropylene, polyester, or Kevlar thread. Nylon threads will not be allowed.

Geotextile Properties

Table 1. Properties for high strength geotextile for embankment reinforcement.

Property	Test Method[1]	Geotextile Property Requirements[2]
AOS	ASTM D4751	0.84 mm max. (#20 sieve)
Water Permittivity	ASTM D4491	0.02/sec. min.
Tensile Strength, min. in machine direction	ASTM D4595	(to be based on project specific design)
Tensile Strength, min. in x-machine direction	ASTM D4595	(to be based on project specific design)
Secant Modulus at 5% strain	ASTM D4595	(to be based on project specific design)
Seam Breaking Strength	ASTM D4884	(to be based on project specific design)
Puncture Resistance	ASTM D4833	330 N min.
Tear Strength, min. in machine and x-machine direction	ASTM D4533	330 N min.
Ultraviolet (UV) Radiation Stability	ASTM D4355	50% Strength Retained min., after 500 Hrs in weatherometer

[1] The test procedures are essentially in conformance with the most recently approved ASTM geotextile test procedures, except geotextile sampling and specimen conditioning, which are in accordance with WSDOT Test Methods 914 an 915, respectively. Copies of these test methods are available at the Olympia Service Center Materials Laboratory in Tumwater, Washington.

[2]All geotextile properties listed above are minimum average roll values (i.e., the test result for any sampled roll in a lot shall meet or exceed the values listed).

Geotextile Approval

Source Approval

The Contractor shall submit to the Engineer the following information regarding each geotextile proposed for use:

Manufacturer's name and current address,
Full Product name,
Geotextile structure, including fiber/yarn type, and
Geotextile polymer type(s).

If the geotextile source has not been previously evaluated, a sample of each proposed geotextile shall be submitted to the Olympia Service Center Materials Laboratory in Tumwater for evaluation. After the sample and required information for each geotextile type have arrived at the Olympia Service Center Materials Laboratory in Tumwater, a maximum of 14 calendar days will be required for this testing. Source approval will be based on conformance to the applicable values from Table 1. Source approval shall not be the basis of acceptance of specific lots of material unless the lot sampled can be clearly identified, and the number of samples tested and approved meet the requirements of WSDOT Test Method 914.

Geotextile Samples for Source Approval

Each sample shall have minimum dimensions of 1.5 meters by the full roll width of the geotextile. A minimum of 6 square meters of geotextile shall be submitted to the Engineer for testing. The geotextile machine direction shall be marked clearly on each sample submitted for testing. The machine direction is defined as the direction perpendicular to the axis of the geotextile roll.

The geotextile samples shall be cut from the geotextile roll with scissors, sharp knife, or other suitable method which produces a smooth geotextile edge and does not cause geotextile ripping or tearing. The samples shall not be taken from the outer wrap of the geotextile nor the inner wrap of the core.

Acceptance Samples

Samples will be randomly taken by the Engineer at the job site to confirm that the geotextile meets the property values specified.

Approval will be based on testing of samples from each lot. A "lot" shall be defined for the purposes of this specification as all geotextile rolls within the consignment (i.e., all rolls sent to the project site) which were produced by the same manufacturer during a continuous period of production at the same manufacturing plant and have the same product name. After the samples and manufacturer's certificate of compliance have arrived at the Olympia Service Center Materials Laboratory in Tumwater, a maximum of 14 calendar days will be required for this testing. If the results of the testing show that a geotextile lot, as defined, does not meet the properties required in Table 1, the roll or rolls which were sampled will be rejected. Two additional rolls for each roll tested which failed from the lot previously tested will then be selected at random by the Engineer for sampling and retesting. If the retesting shows that any of the additional rolls tested do not meet the required properties, the entire lot will be rejected. If the test results from all the rolls retested meet the required properties, the entire lot minus the roll(s) which failed will be accepted. All geotextile which has defects, deterioration, or damage, as determined by the Engineer, will also be rejected. All rejected geotextile shall be replaced at no expense to the Contracting Agency.

Certificate of Compliance
The Contractor shall provide a manufacturer's certificate of compliance to the Engineer which includes the following information about each geotextile roll to be used:

Manufacturer's name and current address,
Full product name,
Geotextile structure, including fiber/yarn type,
Geotextile polymer type(s),
Geotextile roll number, and
Certified test results.

Approval Of Seams
If the geotextile seams are to be sewn in the field, the Contractor shall provide a section of sewn seam which can be sampled by the Engineer before the geotextile is installed.

The seam sewn for sampling shall be sewn using the same equipment and procedures as will be used to sew the production seams. The seam sewn for sampling must be at least 2 meters in length. If the seams are sewn in the factory, the Engineer will obtain samples of the factory seam at random from any of the rolls to be used. The seam assembly description shall be submitted by the Contractor to the Engineer and will be included with the seam sample obtained for testing. This description shall include the seam type, stitch type, sewing thread type(s), and stitch density.

Construction Requirements

Geotextile Roll Identification, Storage, and Handling
Geotextile roll identification, storage, and handling shall be in conformance to ASTM D 4873. During periods of shipment and storage, the geotextile shall be stored off the ground. The geotextile shall be covered at all times during shipment and storage such that it is fully protected from ultraviolet radiation including sunlight, site construction damage, precipitation, chemicals that are strong and acids or strong bases, flames including welding sparks, temperatures in excess of 70^{o} C, and any other environmental condition that may damage the physical property values of the geotextile.

Preparation and Placement of the Geotextile Reinforcement
The area to be covered by the geotextile shall be graded to a smooth, uniform condition free from ruts, potholes, and protruding objects such as rocks or sticks. The Contractor may construct a working platform, up to 0.6 meters in thickness, in lieu of grading the existing ground surface. A working platform is required where stumps or other protruding objects which cannot be removed without excessively disturbing the subgrade are present. All stumps shall be cut flush with the ground surface and covered with at least 150 mm of fill before placement of the first geotextile layer. The geotextile shall be spread immediately ahead of the covering operation. The geotextile shall be laid with the machine direction perpendicular or parallel to centerline as shown in Plans. Perpendicular and parallel directions shall alternate. All seams shall be sewn. Seams to connect the geotextile strips end to end will not be allowed, as shown in the Plans. The geotextile shall not be left exposed to sunlight during installation for a total of more than 14 calendar days. The geotextile shall be laid smooth without excessive wrinkles. Under no circumstances shall the geotextile be dragged through mud or over sharp objects which could damage the geotextile. The cover material shall be placed on the geotextile in such a manner that a minimum of 200 mm of material will be between the equipment tires or tracks and the geotextile at all times. Construction vehicles shall be limited in size and weight such that rutting in the initial lift above the geotextile is not greater than 75 mm deep, to prevent overstressing the geotextile. Turning of

vehicles on the first lift above the geotextile will not be permitted. Compaction of the first lift above the geotextile shall be limited to routing of placement and spreading equipment only. No vibratory compaction will be allowed on the first lift.

Small soil piles or the manufacturer's recommended method shall be used as needed to hold the geotextile in place until the specified cover material is placed.

Should the geotextile be torn or punctured or the sewn joints disturbed, as evidenced by visible geotextile damage, subgrade pumping, intrusion, or roadbed distortion, the backfill around the damaged or displaced area shall be removed and the damaged area repaired or replaced by the Contractor at no expense to the Contracting Agency. The repair shall consist of a patch of the same type of geotextile placed over the damaged area. The patch shall be sewn at all edges.

If geotextile seams are to be sewn in the field or at the factory, the seams shall consist of two parallel rows of stitching, or shall consist of a J-seam, Type Ssn-1, using a single row of stitching. The two rows of stitching shall be 25 mm apart with a tolerance of plus or minus 13 mm and shall not cross, except for restitching. The stitching shall be a lock-type stitch. The minimum seam allowance, i.e., the minimum distance from the geotextile edge to the stitch line nearest to that edge, shall be 40 mm if a flat or prayer seam, Type SSa-2, is used. The minimum seam allowance for all other seam types shall be 25 mm. The seam, stitch type, and the equipment used to perform the stitching shall be as recommended by the manufacturer of the geotextile and as approved by the Engineer.

The seams shall be sewn in such a manner that the seam can be inspected readily by the Engineer or his representative. The seam strength will be tested and shall meet the requirements stated in this Specification.

Embankment construction shall be kept symmetrical at all times to prevent localized bearing capacity failures beneath the embankment or lateral tipping or sliding of the embankment. Any fill placed directly on the geotextile shall be spread immediately. Stockpiling of fill on the geotextile will not be allowed.

The embankment shall be compacted using Method B of Section 2-03.3(14)C. Vibratory or sheepsfoot rollers shall not be used to compact the fill until at least 0.5 meters of fill is covering the bottom geotextile layer and until at least 0.3 meters of fill is covering each subsequent geotextile layer above the bottom layer.

The geotextile shall be pretensioned during installation using either Method 1 or Method 2 as described herein. The method selected will depend on whether or not a mudwave forms during placement of the first one or two lifts. If a mudwave forms as fill is pushed onto the first layer of geotextile, Method 1 shall be used. Method 1 shall continue to be used until the mudwave ceases to form as fill is placed and spread. Once mudwave formation ceases, Method 2 shall be used until the uppermost geotextile layer is covered with a minimum of 0.3 meters of fill. These special construction methods are not needed for fill construction above this level. If a mudwave does not form as fill is pushed onto the first layer of geotextile, then Method 2 shall be used initially and until the uppermost geotextile layer is covered with at least 0.3 meters of fill.

Method 1

After the working platform, if needed, has been constructed, the first layer of geotextile shall be laid in continuous transverse strips and the joints sewn together. The geotextile shall be stretched manually to ensure that no wrinkles are present in the geotextile. The fill shall be end-dumped and spread from the edge of the geotextile. The fill shall first be placed along the outside edges of the geotextile to form access roads. These access roads will serve three purposes: to lock the edges of the geotextile in place, to contain the mudwave, and to provide access as needed to place fill in the center of the embankment. These access roads shall be approximately 5 meters wide. The access roads at the edges of the geotextile shall have a minimum height of 0.6 meters when completed. Once the access roads are approximately 15 meters in length, fill shall be kept ahead of the filling operation, and the access roads shall be kept approximately 15 meters ahead of this filling operation as shown in the Plans. Keeping the mudwave ahead of this filling operation and keeping the edges of the geotextile from moving by use of the access roads will effectively pre-tension the geotextile. The geotextile shall be laid out no more than 6 meters ahead of the end of the access roads at any time to prevent overstressing of the geotextile seams.

Method 2
After the working platform, if needed, has been constructed, the first layer of geotextile shall be laid and sewn as in Method 1. The first lift of material shall be spread from the edge of the geotextile, keeping the center of the advancing fill lift ahead of the outside edges of the lift as shown in the Plans. The geotextile shall be manually pulled taut prior to fill placement. Embankment construction shall continue in this manner for subsequent lifts until the uppermost geotextile layer is completely covered with 0.3 meters of compacted fill.

Measurement
High strength geotextile for embankment reinforcement will be measured by the square meter for the ground surface area actually covered.

Payment
The unit contract price per square meter for "High Strength Geotextile For Embankment Reinforcement", shall be full pay to complete the work as specified.

7.7 COST CONSIDERATIONS

The cost analysis for a geosynthetic reinforced embankment includes:

1. Geosynthetic cost: including purchase price, factory prefabrication, and shipping.
2. Site preparation: including clearing and grubbing, and working table preparation.
3. Geosynthetic placement: related to field workability (see Christopher and Holtz, 1989),
   a) with no working table, or
   b) with a working table.
4. Fill material: including purchasing, hauling, dumping, compaction, allowance for additional fill due to embankment subsidence. (NOTE: Use free-draining granular fill for the lifts adjacent to geosynthetic to provide good adherence and drainage.)

7.8 CONSTRUCTION PROCEDURES

The construction procedures for reinforced embankments on soft foundations are extremely important. *Improper* fill placement procedures can lead to geosynthetic damage, nonuniform settlements, and even embankment failure. By the use of low ground pressure equipment, a properly selected geosynthetic, and proper procedures for placement of the fill, these problems can essentially be eliminated. Essential construction details are outlined below. The Washington State DOT Special Provision in Section 7.6 provides additional details.

A. Prepare subgrade:
   1. Cut trees and stumps flush with ground surface.
   2. Do not remove or disturb root or meadow mat.
   3. Leave small vegetative cover, such as grass and reeds, in place.
   4. For undulating sites or areas where there are many stumps and fallen trees, consider a working table for placement of the reinforcement. In this case, a lower strength sacrificial geosynthetic designed only for constructability can be used to construct and support the working table.

B. Geosynthetic placement procedures:
   1. Orient the geosynthetic with the machine direction perpendicular to the embankment alignment. No seams should be allowed parallel to the alignment. Therefore,
      - The geosynthetic rolls should be shipped in unseamed machine direction lengths equal to one or more multiples of the embankment design base width.
      - The geosynthetic should be manufactured with the largest machine width possible.
      - These widths should be factory-sewn to provide the largest width compatible with shipping and field handling.

   2. Unroll the geosynthetic as smoothly as possible transverse to the alignment. (Do not drag it.)

   3. Geotextiles should be sewn as required with all seams up and every stitch inspected. Geogrids should be positively joined by clamps, cables, pipes, etc.

   4. The geosynthetic should be manually pulled taut to remove wrinkles. Weights (sand bags, tires, etc.) or pins may be required to prevent lifting by wind.

5. Before covering, the Engineer should examine the geosynthetic for holes, rips, tears, etc. Defects, if any, should be repaired by.
   - Large defects, should be replaced by cutting along the panel seam and sewing in a new panel.
   - Smaller defects, can be cut out and a new panel resewn into that section, if possible.
   - Defects less than 150 mm, can be overlapped a minimum of 1 m or more in all directions from the defective area. (Additional overlap may be required, depending on the geosynthetic-to-geosynthetic friction angle).

   NOTE: If a *weak link* exists in the geosynthetic, either through a defective seam or tear, the system will tell the engineer about it in a dramatic way -- spectacular failure! (Holtz, 1990)

C. Fill placement, spreading, and compaction procedures:

1. Construction sequence for extremely soft foundations (when a mudwave forms) is shown in Figure 7-6.
   a. End-dump fill along edges of geosynthetic to form toe berms or access roads.
      - Use trucks and equipment compatible with constructability design assumptions (Table 7-1).
      - End-dump on the previously placed fill; do not dump directly on the geosynthetic.
      - Limit height of dumped piles, *e.g.*, to less than 1 m above the geosynthetic layer, to avoid a local bearing failure. Spread piles immediately to avoid local depressions.
      - Use lightweight dozers and/or front-end loaders to spread the fill.
      - Toe berms should extend one to two panel widths ahead of the remainder of the embankment fill placement.
   b. After constructing the toe berms, spread fill in the area between the toe berms.
      - Placement should be parallel to the alignment and symmetrical from the toe berm inward toward the center to maintain a *U*-shaped leading edge (concave outward) to contain the mudwave (Figure 7-7).

c. Traffic on the first lift should be parallel to the embankment alignment; no turning of construction equipment should be allowed.
    - Construction vehicles should be limited in size and weight to limit initial lift rutting to 75 mm. If rut depths exceed 75 mm, decrease the construction vehicle size and/or weight.

d. The first lift should be compacted only by *tracking in place* with dozers or end-loaders.

e. Once the embankment is at least 600 mm above the original ground, subsequent lifts can be compacted with a smooth drum vibratory roller or other suitable compactor. If localized liquefied conditions occur, the vibrator should be turned off and the weight of the drum alone should be used for compaction. Other types of compaction equipment also can be used for nongranular fill.

2. After placement, the geosynthetic should be covered within 48 hours.

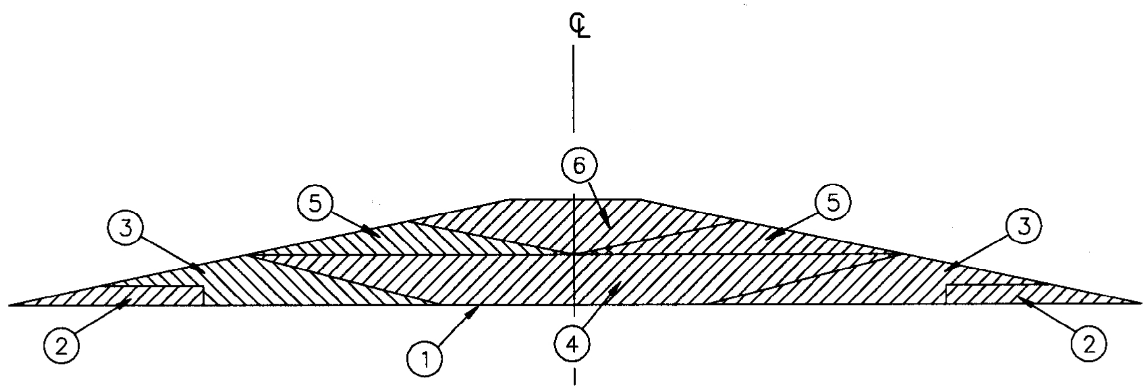
   For less severe foundation conditions (*i.e.*, when no mudwave forms):
   a. Place the geosynthetic with no wrinkles or folds; if necessary, manually pull it taut prior to fill placement.
   b. Place fill symmetrically from the center outward in an inverted *U* (convex outward) construction process, as shown in Figure 7-8. Use fill placement to maintain tension in the geosynthetic.
   c. Minimize pile heights to avoid localized depressions.
   d. Limit construction vehicle size and weight so initial lift rutting is no greater than 75 mm.
   e. Smooth-drum or rubber-tired rollers may be considered for compaction of first lift; however, do not overcompact. If weaving or localized quick conditions are observed, the first lift should be compacted by tracking with construction equipment.

D. Construction monitoring:
1. Monitoring should include piezometers to indicate the magnitude of excess pore pressure developed during construction. If excessive pore pressures are observed, construction should be halted until the pressures drop to a predetermined safe value.

2. Settlement plates should be installed at the geosynthetic level to monitor settlement during construction and to adjust fill requirements appropriately.

3. Inclinometers should be considered at the embankment toes to monitor lateral displacement.

Photographs of reinforced embankment construction are shown in Figure 7-9.



SEQUENCE OF CONSTRUCTION

1. LAY GEOSYNTHETIC IN CONTINUOUS TRAVERSE STRIPS, SEW STRIPS TOGETHER.
2. END DUMP ACCESS ROADS.
3. CONSTRUCT OUTSIDE SECTIONS TO ANCHOR GEOSYNTHETIC.
4. CONSTRUCT OUTSIDE SECTION TO "SET" GEOSYNTHETIC.
5. CONSTRUCT INTERIOR SECTIONS TO TENSION GEOSYNTHETIC.
6. CONSTRUCT FINAL CENTER SECTION

Figure 7-7. Construction sequence for geosynthetic reinforced embankments for extremely weak foundations (from Haliburton, Douglas and Fowler, 1977).

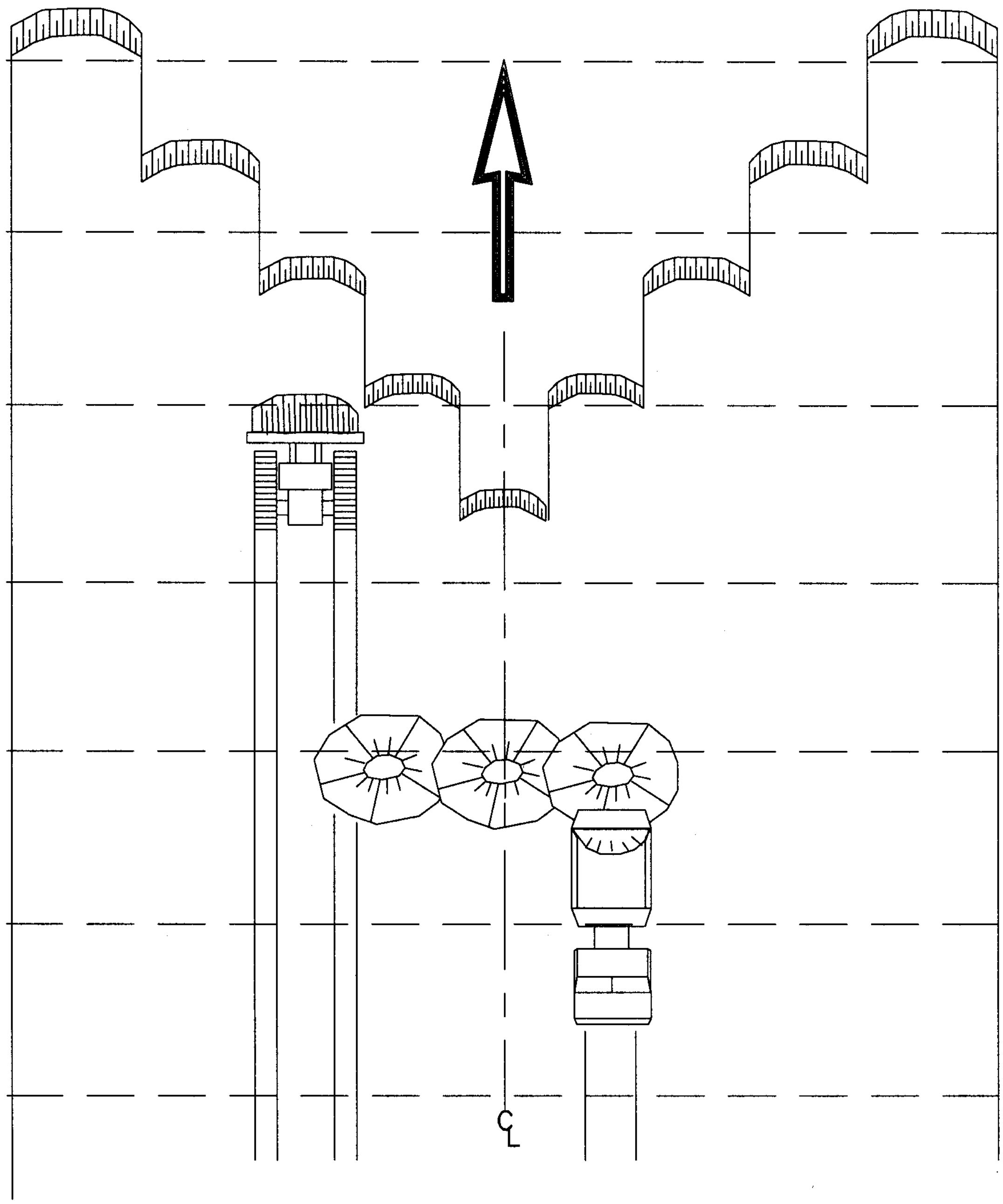


Figure 7-8. Placement of fill between toe berms on extremely soft foundations (CBR < 1) with a mud wave anticipated

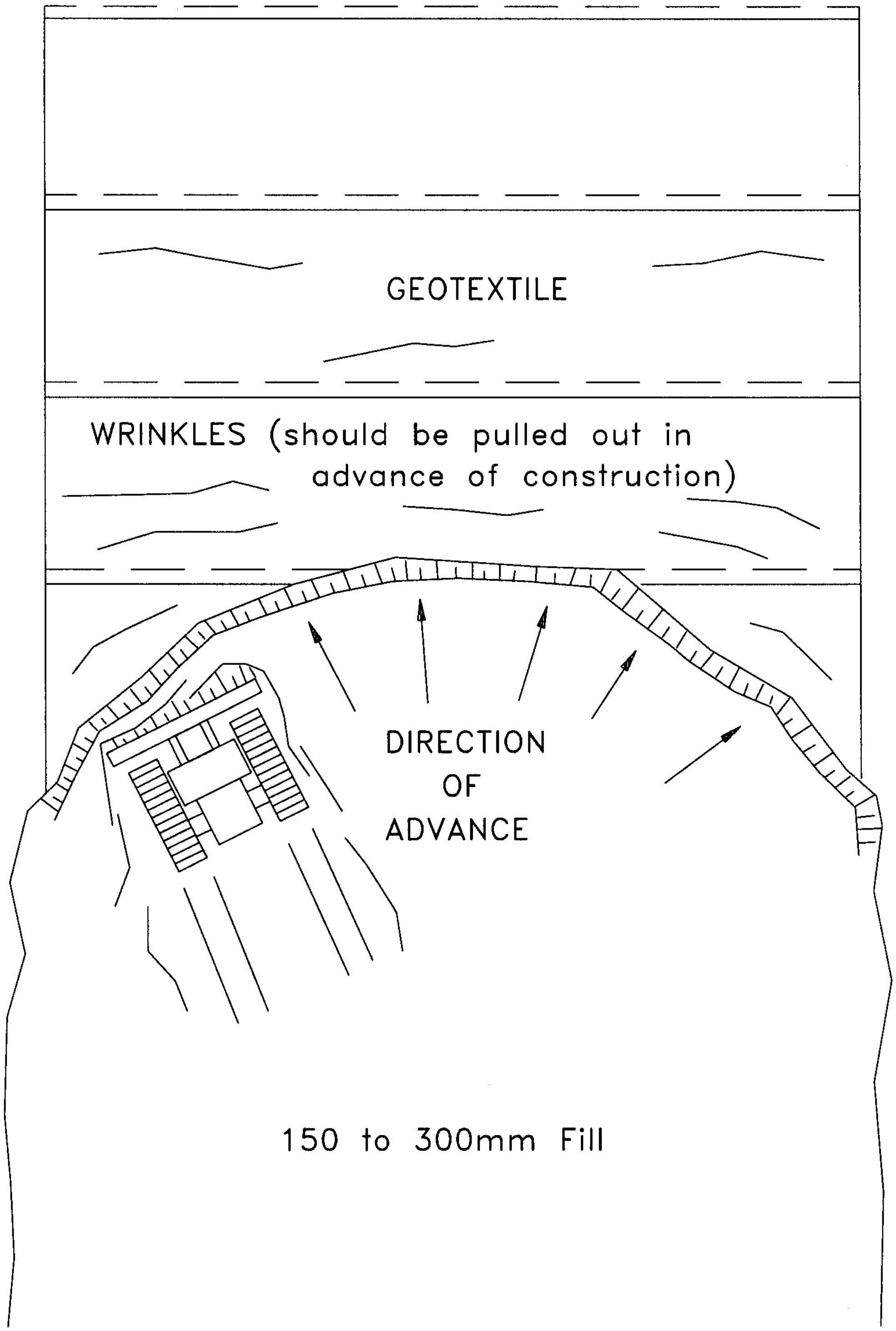


Figure 7-9. Fill placement to tension geotextile on moderate ground conditions; moderate subgrade (CBR > 1); no mud wave.

(a)

(b)

(c)

Figure 7-10. Reinforced embankment construction; a) geosynthetic placement; b) fill dumping; and c) fill spreading.

7.9 INSPECTION

Since implemented construction procedures are crucial to the success of reinforced embankments on very soft foundations, competent and professional construction inspection is absolutely essential. Field personnel must be properly trained to observe every phase of the construction and to ensure that (1) the specified material is delivered to the project, (2) the geosynthetic is not damaged during construction, and (3) the specified sequence of construction operations are explicitly followed. Field personnel should review the checklist in Section 1.7.

7.10 REINFORCED EMBANKMENTS FOR ROADWAY WIDENING

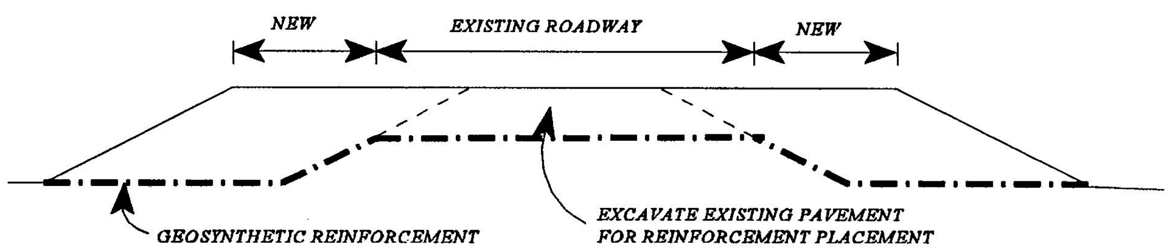
Special considerations are required for widening of existing roadway embankments founded on soft foundations. Construction sequencing of fill placement, connection of the geosynthetic to the existing embankment, and settlements of both the existing and new fills must be addressed by the design engineer. Analytical techniques for geosynthetic reinforcement requirements are the same as those discussed in Section 7.3.

Two example roadway widening cross sections are illustrated in Figure 7-10. The addition of a vehicle lane on either side of an existing roadway (Figure 7-10a) is feasible if the traffic can be detoured during construction. In this case, the reinforcement may be placed continuously across the existing embankment and beneath the two new outer fill sections. Placing both new lanes to one side of the embankment (Figs. 7-10b) may allow for maintaining one lane of traffic flow during construction. With the new fill placed to one side of the existing embankment, the anchorage of the geosynthetic into the existing embankment becomes an important design step.

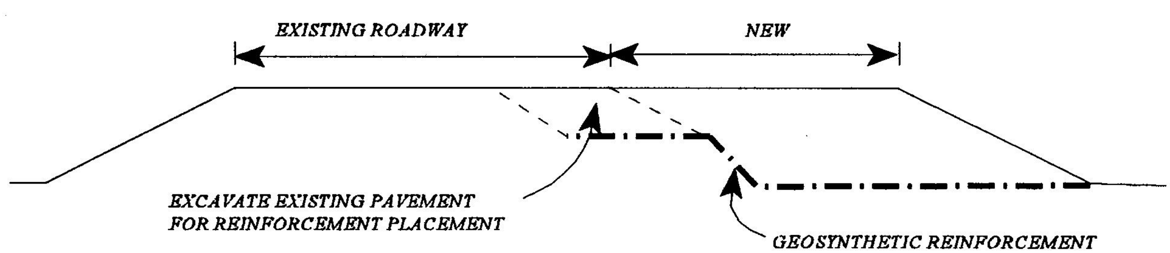
Both the new fill sections and the existing fill sections will most likely settle during and after fill placement, although the amount of settlement will be greater for the new fill sections. The existing fills settle because of the influence of the new, adjacent fill loads on their foundation soils. The amount of settlements is a function of the foundation soils and amount of load (fill height). When fill is placed to one side of an embankment (Figs. 7-10b) the pavement may need substantial maintenance during construction and until settlements are nearly complete. Alternatively, light-weight fill could be used to reduce the settlement of the new fill and existing sections.

Note that the sections in Figure 7-10 do not indicate a geosynthetic reinforcement layer beneath the existing embankment section. Typically, the reinforcement for the embankment widening section would be designed assuming no contribution of existing section geosynthetic in reinforcing the new and combined sections. Therefore, connection of the new reinforcement to any existing reinforcement is normally not required.

For soft subgrades, where a mud wave is anticipated, construction should be parallel to the alignment with the outside fill placed in advance of the fill adjacent to the existing embankment. For firm subgrades, with no mudwave, fill may be placed outward, perpendicular to the alignment.



(a)



(b)

Figure 7-11. Reinforced embankment construction for roadway widening; a) fill placement on both sides of existing embankment; b) fill placement on one side of the existing fill.

7.11 REINFORCEMENT OF EMBANKMENTS COVERING LARGE AREAS

Special considerations are required for constructing large reinforced areas, such as parking lots, toll plazas, storage yards for maintenance materials and equipment, and construction pads. Loads are more biaxial than conventional highway embankments, and design strengths and strain considerations must be the same in all directions. Analytical techniques for geosynthetic reinforcement requirements are the same as those discussed in Section 7.3. Because geosynthetic strength requirements will be the same in both directions, including across the seams, special seaming techniques must often be considered to meet required strength requirements. Ends of rolls may also require butt seaming. In this case, rolls of different lengths should be used to stagger the butt seams. Two layers of fabric should be considered, with the bottom layer seams laid in one direction, and the top layer seams laid perpendicular to the bottom layer. The layers should be separated by a minimum lift thickness, usually 300 mm, soil layer.

For extremely soft subgrades, the construction sequence must be well planned to accommodate the formation and movement of mudwaves. Uncontained mudwaves moving outside of the construction can create stability problems at the edges of the embankment. It may be desirable to construct the fill in parallel embankment sections, then connect the embankments to cover the entire area. Another method staggers the embankment load by constructing a wide, low embankment with a higher embankment in the center. The outside low embankments are constructed first and act as berms for the center construction. Next, an adjacent low embankment is constructed from the outside into the existing embankment; then the central high embankment is spread over the internal adjacent low embankment. Other construction schemes can be considered depending on the specific design requirements. In all cases, a perimeter berm system is necessary to contain the mudwave.

7.12 COLUMN SUPPORTED EMBANKMENTS

An alternate approach of embankment construction on soft soils may be used when time constraints are critical to the success of the project. Column supported embankments (CSE) with a geosynthetic reinforced load transfer platform are designed to transfer the load of the embankment through the soft compressible soil layer to a firm foundation, thus eliminating the construction wait time for dissipation of pore water pressures and minimizing settlement of the foundation soils. This technology was first used in Sweden in 1971, and has been used successfully on projects in the U.S. since 1994.

The load from the embankment must be effectively transferred to the columns to prevent punching of the columns through the embankment fill causing differential settlement at the surface of the embankment. If the columns are placed close enough together, soil arching will occur and the load will be transferred to the columns. A "conventional" CSE is where the columns are spaced relatively close together, and some battered columns are used at the sides of the embankment to prevent lateral spreading. In order to minimize the number of columns required to support the embankment and increase the efficiency of the design, a geosynthetical reinforced load transfer platform (LTP) may be used. The load transfer platform consists of one or more layers of geosynthetic reinforcement placed between the top of the columns and the bottom of the embankment. A CSE with geosynthetic reinforcement is schematically shown in Figure 7-11.

The key advantage to CSE is that construction may proceed rapidly in one stage. One major benefit of CSE technology is that it is not limited to any one-column type. Where the infrastructure precludes high-vibration techniques, the type of column used for the CSE system may be selected to minimize or eliminate the potential for vibrations. If contaminated soils are anticipated at a site, the column type may be selected so that there are no spoils from the installation process. The designer has the flexibility of selection of the most appropriate column for the project. Total and differential settlement of the embankment may be drastically reduced when using CSE over conventional approaches.

A major disadvantage of CSE is often initial construction cost when compared to other solutions. However, if the time savings when using CSE technology is included in the economic analysis, the cost may be far less than other solutions. Another major disadvantage is that there is currently no single well-accepted design procedure. There are many different design approaches, and they all give different results. Design procedures and recommendations are presented in FHWA NHI-04-001, Ground Improvement Methods (Elias et al., 2004) – the reference manual used with the 3-day NHI Ground Improvement Course #132034.

Applications where CSE technology is appropriate for transportation include

- embankment stabilization
- roadway widening
- bridge approach fill stabilization
- bridge abutment and other foundation support

A considerable amount of highway widening and reconstruction work will be required in future years. Some of this work will involve building additional lanes immediately adjacent to existing highways constructed on moderate to high fills over soft cohesive soils, such as those found in wetland areas. For this application, differential settlement between the existing and new construction is an important consideration, in addition to embankment stability. Support of the new fill on CSE offers a viable design alternative to conventional construction.

CSE may be used whenever an embankment must be constructed on soft compressible soil. To date, the technology has been limited to embankment heights in the range of 10 meters (33 ft). CSE technology reduces post construction settlements of the embankment surface to typically less than 50-100 mm (2-4 in.). A generalized summary of the factors that should be considered when assessing the feasibility of utilizing CSE technology on a project is presented in FHWA NHI-04-001, Ground Improvement Methods.

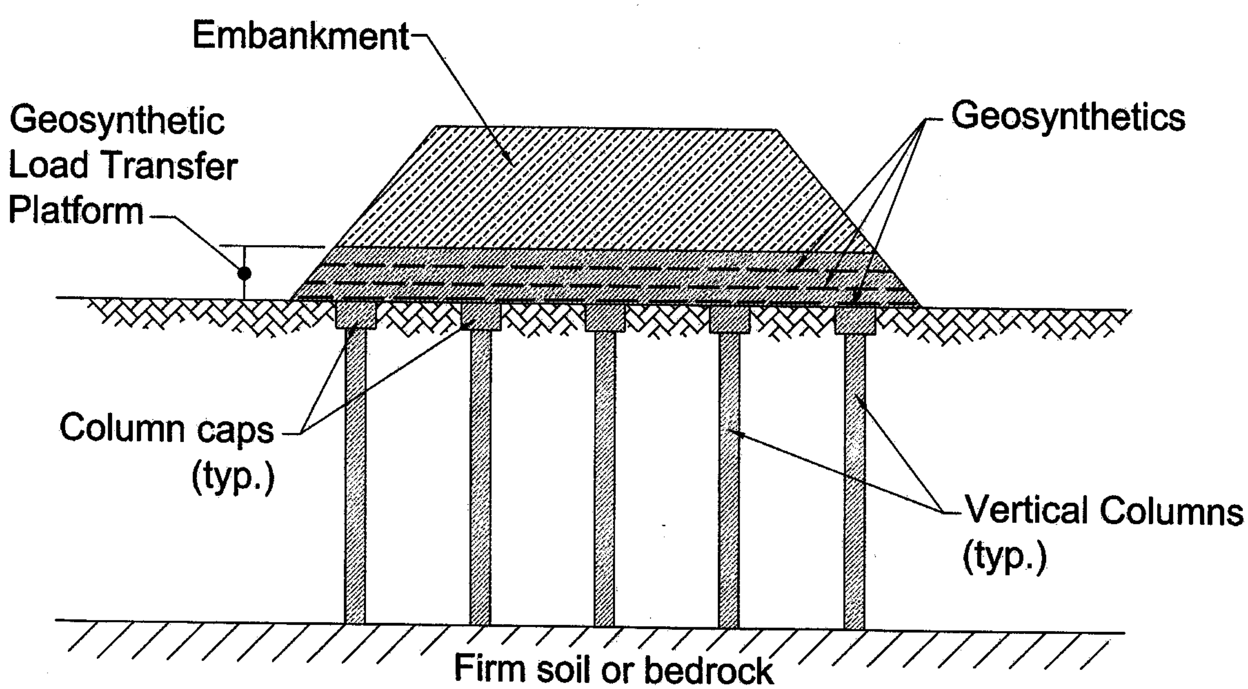


Figure 7-12. Column Supported Embankment with Geosynthetic Reinforcement.

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APPENDIX

LISTS OF
GROUND IMPROVEMENT
CONTRACTORS

LIST FROM

ADSC

COMPANY	FULL_NAME	TITLE	ADDRESS_1	CITY	STATE/ PROV.	ZIP	EMAIL
GROUTING							
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Gil's Drilling & Excavating, Inc.	Raymond Suiter	President	12473 N. 92nc	Peoria	AZ	85381	rsuiter@gilsdrilling.com
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TCDI (The Concrete Doctor, Inc.)	Steven D. Scherer	President	1477 Barclay E	Buffalo Grove	IL	60089-4537	steve@geomod.com
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Subsurface Construction Company, LLC	Kelly Williams	Superintendent	1107 Fuller St	Raleigh	NC		27603	kelly@subsurfaceconstruction.com
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Drainage & Ground Improvement, Inc. Mr. Seth Pearlman President 275 Millers Run Road Bridgeville, PA 15017 1-800-326-6015 **Terra Systems** Mr. John Jones 39565 Cottage Grove Lane Lovettsville, VA 20180 (540) 882-4130

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.</td>
<td>Terra Systems
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(540) 882-4130</td>
<td>Drainage & Ground Improvement, Inc.
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1-800-326-6015</td>
</tr>
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<table>
<tr><th colspan="3">SOIL NAIL WALLS</th></tr>
<tr>
<td>Brayman Foundation Division
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724-443 1533

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<td>Richard Goettle, Inc.
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Hayward Baker, Inc.
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410-551-8200</td>
<td>Nicholson Construction Company
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12 McClane St.
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39 Utter Avenue
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973-636-2700</td>
</tr>
</table>

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DEEP SOIL MIXING		
Condon Johnson & Associates Inc. Attn: Mr. James Johnson 1840 Embarcadero Oakland, CA 94606 510-534-3400 **GeoCon** **Hayward Baker, Inc.** Attn: Mr. George K. Burke, P.E. Vice President - Engineering 1130 Annapolis Road Odenton, MD 21113 410-551-8200 **Trevi-ICOS Corporation/Hercules** Dr. Arturo L. Ressi di Cervia President 250 Summer Street Boston, MA 02210 617-345-9955	**Malcolm Drilling Company, Inc.** Attn: Mr. John Roe 425 Barneveld Avenue San Francisco, CA 94124-1501 (415) 642-5353 **Raito, Inc.** Attn: Dr. David S. Yang 1660 Factor Ave. San Leandro, CA 94577 510-346-9840 **Remedial Construction Services, Inc.** Attn: Mr. Chris McGhee 2571 Merrell Rd. Dallas, TX 75229 972-241-7016 **Schnabel Foundation Company** Mr. David E. Weatherby Vice President 45240 Business Court, Suite 250 Sterling, VA 20166 703-742-0020	**Underpinning & Foundation Constructors, Inc.** Attn: Mr. Edward Forte, P.E. 46-36 54th Road Maspeth, NY 11378 718-786-6557

GROUTING

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Hayward Baker, Inc.
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Attn: Mr. Lynn Crayne
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