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GROUND IMPROVEMENT TECHNICAL SUMMARIES

VOLUME I

Demonstration Project 116

Working Draft: September 1998

TECHNOLOGY APPLICATIONS
Innovation Through Partnerships

SI CONVERSION FACTORS AND BASE UNITS

Quantity	From English Units	To SI Units	[6]Multiply By
Length	mile yard foot inch	km m m mm	1.609 0.9144 0.3048 25.40
Area	square mile acre acre square yard square foot square inch	km^2 m^2 hectare m^2 m^2 mm^2	2.590 4047 0.404 0.836 0.092 645.2
Volume	acre foot cubic yard cubic foot cubic foot 1000 board feet gallon cubic inch	m^3 m^3 m^3 L (1000 cm^3) m^3 L (1000 cm^3) cm^3	1233 0.764 0.028 28.32 2.36 3.785 16.39
Mass	pound mass, lb-mass, lbm	kg	0.4536
Mass Density	lb-mass/ft^3	kg/m^3	16.02
Force	lb kip	N kN	4.448 4.448
Force/Unit Length	lbs/ft kips/ft	N/m kN/m	14.59 14.59
Force/Unit Area, Pressure, Stress, Modulus of Elasticity.	lbs/in^2 kips/in^2 lbs/ft^2 kips/ft^2	kPa MPa Pa kPa	6.895 6.895 47.88 47.88
Force/Volume Unit Weight	lbs/ft^3 kips/ft^3	N/m^3 kN/m^3	157.1 157.1
Bending Moment, Torque, Moment of force	ft-lb ft-kip	N·m kN·m	1.356 1.356

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

Conversion Factors Continued

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

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

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FHWA Technical Consultant - Jerry A. DiMaggio

16. Abstract

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

The technical summaries reflect current practice in design, construction, contracting methods and quality procedures. The individual technical summaries are intended to serve as a primer on a particular subject; containing an executive summary of technical information which provides concise and specific guidance to users who desire further information. Technical summaries are not intended to impart a working knowledge of the technology, assess its applicability costs, contracting methods to specific projects.

17. Key Words MECHANICALLY STABILIZED EARTH WALLS, REINFORCED SOIL SLOPES, SOIL NAILING, WICK DRAINS, LIGHTWEIGHT FILL, DYNAMIC COMPACTION, DEEP SOIL STABILIZATION, GROUTING, STONE COLUMNS, VIBROCOMPACTION	18. Distribution Statement No restrictions.

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NOTICE

PREFACE

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

In order to take advantage of these new developments, the FHWA has developed these Technical Summaries in connection with Demonstration Project No. 116, Ground Improvement. The primary purpose of the summaries is to support educational programs conducted by FHWA for transportation agencies. This program consists of (1) a workshop for geotechnical, structural, roadway and construction engineers and (2) technical assistance for project development in areas covered by this Demonstration Project on request to transportation agencies.

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

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

- *Mechanically Stabilized Earth Walls and Reinforced Soil Slopes Design and Construction Guidelines,* by V. Elias and B. Christopher, FHWA-SA-96-071.

- *Manual for Design and Construction Monitoring of Soil Nail Walls,* by R.J. Byrne, D. Cotton, J.Portefield, C. Walschlag and G. Ueblaker, FHWA-SA-96-069.

- *Dynamic Compaction, Geotechnical Engineering Circular No. 1,* by R. Lukas, FHWA-SA-95-037.

- *Prefabricated Vertical Drains, Vol. 1,* by J.J. Rixner, S.R. Kraemer and A.D. Smith, FHWA-RD-86-168.

- *Design and Construction of Stone Columns, Vol. 1,* by R. Barkdale and R. Bachus, FHWA-RD-83-02C.

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

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

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

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

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

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INTRODUCTION TO TECHNICAL SUMMARIES

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

Ground improvement has the following main functions:

- To increase bearing capacity,
- to control deformations and accelerate consolidation,
- to decrease imposed loads,
- to provide lateral stability,
- to form seepage cutoffs or fill voids and,
- to increase resistance to liquefaction.

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

1. Consolidation
   - by Wick Drains, section 1.

2. Load Reduction
   - by use of Lightweight Fills, section 2.

3. Densification
   - by Vibrocompaction, section 3.
   - by Dynamic Compaction, section 4.

4. Reinforcement
   - by Mechanically Stabilized Earth and Reinforced Soil Slopes, section 5.
   - by Soil Nailing, section 6.
   - by Stone Columns, section 7.

5. Chemical Stabilization
   - by Soil Mixing, section 8.
   - by Grouting, section 9.

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

For the geotechnical specialist, state-of-the-art design and construction references are provided.

WICK DRAINS

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CHAPTER 1

DESCRIPTION AND HISTORY

1.1 DESCRIPTION

Wick drains can be best described as prefabricated vertical drains. Their primary use is to accelerate consolidation of soft saturated compressible soils under load. In simple terms, they greatly decrease the settlement time of embankment over soft soils. By doing so, they also accelerate the rate of strength gain of the in-situ soft soils.

Wick drains are band shaped (rectangular crossection) products consisting of a geotextile jacket surrounding a plastic core. This construction permits porewater in the soil to seep in the drain for collection and transmittal up and down the length of the core. Basic evolution of the term wick drains comes from the idea that they look like a wick, and the first available product was known as a "cardboard wick." Other common terms for wick drains are prefabricated vertical or PV drains, drainage wicks, band drains and strip drains (although the term strip drain also refers to other prefabricated composite drains).

While there are some variations, the size of a wick drain is typically 100 mm wide by 3 to 9 mm thick. The material consists of a plastic core formed to create channels that are wrapped in a geotextile filter.

Wick drains are only one general type of vertical drain system, the others being sand drains and "sand wicks" or geotextile encased sand drains. The general principles that govern all vertical drain installations are similar to all types of drains. However, the cost advantages of wick drains over other vertical drain systems have resulted in their almost exclusive use except in unusual circumstances.

The most common use of wick drains is to accelerate consolidation for approach embankments at bridges or other embankment construction over soft soils, where the total postconstruction settlement is not acceptable. A typical cross-section of a wick drain installation for embankments is shown in figure 1.

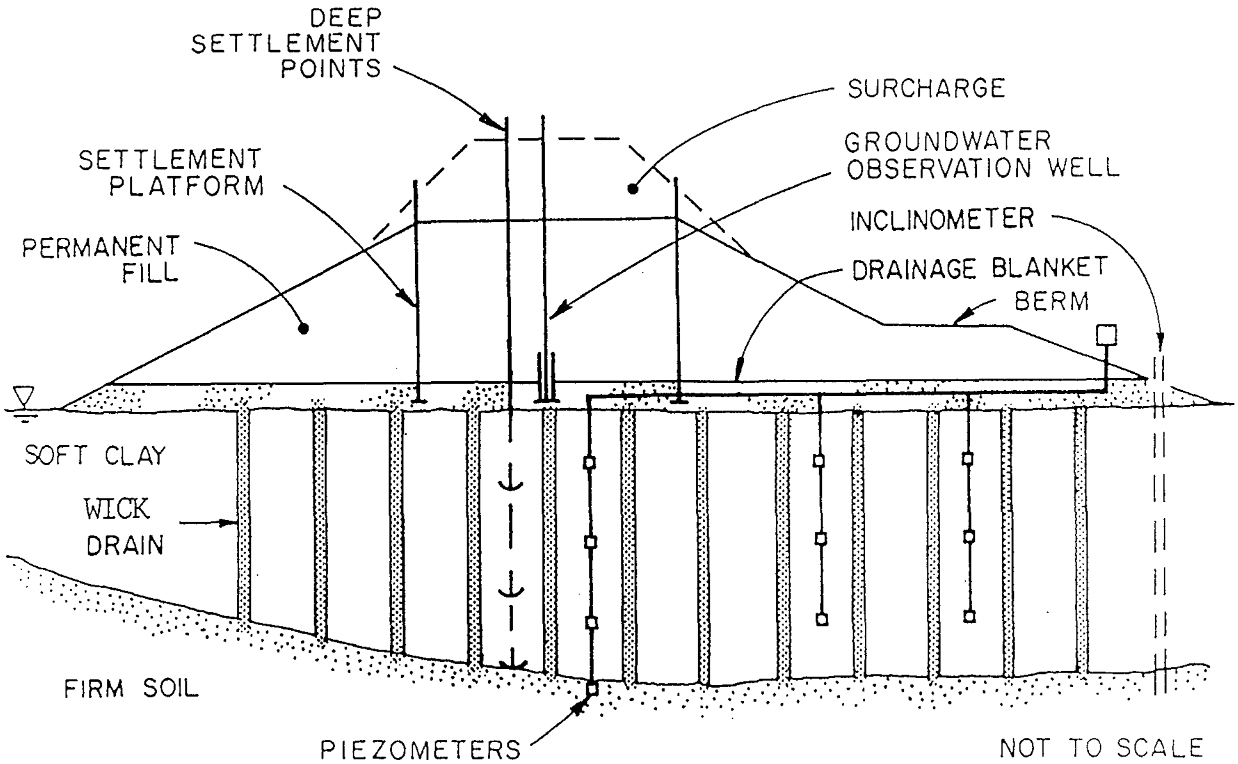


Figure 1. Wick drain instllation for a highway embankment.

1.2 HISTORICAL OVERVIEW

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

In Europe, the lack of available sand with suitable drainage characteristics and certain environmental drawbacks led to the development of the first wick drain in the late 1930s by Walter Kjellman, then Director of The Swedish Geotechnical Institute. This drain utilized three layers of cardboard with the two outer layers serving as a filter and the middle layer as a 10 channel separator; the drain measured 100 mm by 3 mm and became known as the "Cardboard Wick". This drain was used extensively in Sweden after 1939, when a machine for installation was developed. Later these drains found their way into other countries.

Sweden subsequently, however, reverted to the use of sand drains, mainly because of the low permeability, poor resistance to pressure, and the low water transportation capability of the cardboard material. In spite of this, Oleg Wager, a colleague of Kjellman's at The Swedish Geotechnical Institute, believed so strongly in the potential of the wick drain that he worked on and developed an improved drain, and in 1972 patented the Geodrain. This drain had essentially the same dimensions as the "Cardboard Wick" but was constructed differently. The Geodrain consisted of a plastic core containing 27 grooved channels for water transportation and was surrounded by a filter material made of paper. Prefabricated wick drains, as we know them today, had finally arrived. Almost simultaneously the Castleboard Drain was developed in Japan. It was very similar in appearance to the Geodrain, but with a geosynthetic material instead of paper used as a filter.

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

With environmental and economic factors slowly eliminating the use of jetted and augured sand drains, a newly designed wick drain material known as Alidrain was introduced in the United States. Acceptance, while initially slow due to the lack of prior experience and design procedures, grew quite rapidly. Today there are as many as 200 projects completed yearly. In fact, wick drains are now used almost exclusively where a vertical drain solution is required.

Since the development of the first cardboard wick, there have been over 50 different wick drains used worldwide and at least 10 in the United States. Today, most of those installed in the United States are one of two basic designs. Both have an independent outside geosynthetic filter but the core of one type is corrugated and the other is studded. The corrugated design results in straight channels for flow. This type of wick drain design presently accounts for over 90 percent of the United States market. Commonly used brand names are known as AMERDRAIN 407, MEBRA-DRAIN 7407, MEBRA-DRAIN MD 88 and ALIWICK.

The studded core type design results in a more turbulent flow pattern, thereby necessitating a slightly thicker drain in order to obtain the same flow rates. These devices account for 10 percent of the United States market. Common studded core brand names are ALIDRAIN and AMERDRAIN 417.

There are no available data from completed projects that would indicate any advantages or disadvantages by the use of the different types of cores. Much of the early usage was with the studded design, but presently the grooved design is used most often.

Representative wick drains in use are shown in figure 2.

1.3 FOCUS AND SCOPE

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

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

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

3. "In Situ Soil Improvement Techniques," AASHTO-AGC-ARTBA Joint Committee, Subcommittee on New Highway Materials, Task Force 27 Report.

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

The content of this technical summary includes the following:

- Application
- Feasibility
- Construction procedures and monitoring
- Specifications
- Typical case histories

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

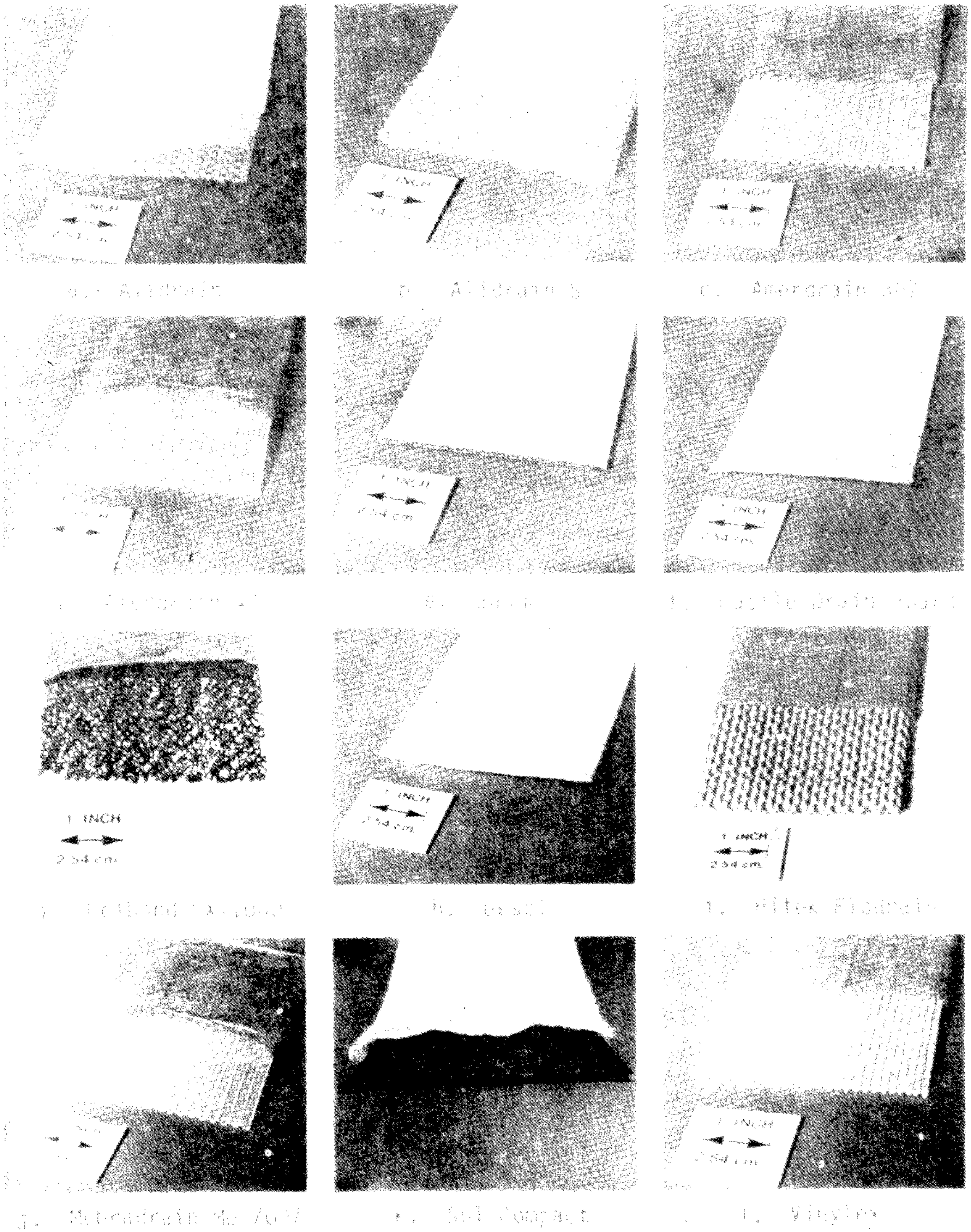


Figure 2. Typical wick drain products.

When used in conjunction with surcharging or preloading, the principal benefits of using wick drains are:

- To decrease the settlement time required such that final construction can be completed in a reasonable time with minimal post construction settlement.

- To decrease the amount of surcharge or preload material required to achieve a settlement in the given time.

- To increase the rate of strength gain due to consolidation of soft soils when stability is of concern.

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

CHAPTER 2

DESIGN CONSIDERATIONS

2.1 APPLICATIONS

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

In general, soils for which wick drains are being used must be saturated and normally to slightly overconsolidated prior to loading. The loading should exceed the maximum past consolidation pressure for the wick drains to be totally effective.

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

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

Special application, liquefaction reduction

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

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

Table 1. Uses of wick drains for transportation applications.

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

2.2 ADVANTAGES AND DISADVANTAGES

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

a. Advantages

Economy

For typical projects, the cost for wick drains will be one-fifth to one-tenth that of 300 to 450 mm sand drains. Site conditions could effect this differential, but more than likely would effect the pricing of both type drains. For projects that are extremely small (less than 1,500 meters), sand drains may be more economical due to high mobilization charges for wick drain equipment.

Installation

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

Continuity of Drain

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

Minimal Displacement

The typical size of mandrel used for wick drains is small enough to create minimal displacement during installation. However, when compared with high quality non-displacement sand drains, wick drains might not be quite as effective. The amount of displacement can be taken into account in the wick drain design process which leads to an increase of time needed for consolidation. There has been no evidence of a significant reduction of shear value of in-situ soils due to remolding as compared with displacement sand drains.

Improved Quality Control

Compared to sand drains, the quality control for wick drain construction is quite simple. Because continuity of the drain is assured during installation, the major duties of inspection are to ensure proper drain anchorage and that they reach required depths.

Equipment Flexibility

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

Less Material Storage

Wick drains come in reels usually containing 140 to 300 meters. Each roll is approximately 1 meter in diameter and 100 mm thick and can be easily stored. Wick drains will eliminate the cost of large sand stock piles and truck traffic usually associated with sand drains.

No Spoil Removal

With the exception of situations where preaugering or drilling is necessary to penetrate to the soft compressible soil layer, there is no significant excess spoil material to be removed from the site. By comparison with jetted sand drains, there is no need to handle and dispose of contaminated water or jetted spoil materials.

No Water Required

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

b. Disadvantages

Greater Quantities

Because of their size, wick drains need to be placed at closer spacings than sand drains. A rule of thumb is that the quantity of wick drains to cover the same area will be 2 to 3 times that of 300-mm mandrel driven sand drains. Even with this differential, the total cost of a wick drain solution is usually substantially less than sand drains as the installation rate on an area basis is much greater.

No Compressive Strength

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

Mobilization Charges

Due to specialized equipment, there is generally a mobilization charge of $8,000 to $10,000 (1996) for each wick drain project. This charge includes transportation to the project site, and time for setup and dismantling. Most sand drain installation equipment would be equal or greater in cost. However, for short drains where local augering equipment could be used, the mobilization cost for sand drains might be less.

Headroom Limitations

Generally, wick drain installation equipment must be 1.5 meters to 3 meters taller than the depth of installation. Headroom requirements for sand drains is at least 3 meters more than the depth of installation.

Material Must be Stored Properly

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

2.3 FEASIBILITY EVALUATIONS

When wick drains 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

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

a. Environmental Considerations

If the in-situ soils are contaminated with any kind of hazardous waste or material, then it is possible that the excess pore water draining through the wick drain will need to be collected and treated. In such situations, care must be exercised in *not allowing* the wick drain to penetrate into a highly permeable layer, should one exist below the compressible stratum.

b. Site Conditions

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

- Uneven working surface
- Limited headroom
- Obstructions above the compressible layer
- Unstable working surface
- Depth of wick drains in excess of 45 m
- Stiff to very stiff compressible layers
- Extremely soft layer for anchoring
- Poor site accessibility
- Overhead or subsurface utility interference

Uneven working surface. Wick drains cannot be installed economically on steep slopes. Therefore, the area will have to be benched with widths sufficient to allow for the equipment. Generally, a minimum bench width of 8 meters is required.

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

Limited headroom. A rule of thumb is that the depth of a wick drain needs to be 3 meters shorter than the available headroom in order to be economically installed. Limited headroom situations occur most often when installing under an existing bridge. Wick drains can be installed vertically in segments with limited headroom, but the cost would most likely be as high as five times the normal unit installation price.

Obstructions above the compressible layer. Where obstructions must be penetrated above the compressible layer, considerable extra costs could be involved. A stiff or dense upper layer can be penetrated without predrilling and will add to the cost. However, obstructions such as concrete, rock, rubble, slag, brick, wood, riprap, stone, debris, rubbish, or trash can result in very expensive predrilling costs.

Unstable working surface. In general, most unstable working surfaces can be made stable prior to the installation of wick drains with the use of geotextiles and granular soil for the drainage layer (600 to 900 mm thick). The installation equipment will usually penetrate these materials without difficulty. Where the ground cannot be stabilized, movable timber mats may be used in support, or specialized lightweight equipment is available at a substantial increase in the unit cost.

Extreme depth. Wick drains have been installed to depths of 60 meters with the use of specialized equipment. A rule of thumb is that wick drains over 18 meters in depth will require a crane for installation. Depths over 35 meters require a very large crane and specialized installation masts.

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

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

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

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

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

2.4 LIMITATIONS

It is important to remember that a wick drain serves no structural function (except perhaps in liquefaction reduction). By providing a shorter drainage path, wick drains provide a faster release of excess pore water, thereby resulting in faster settlement and quicker strength gain by consolidation. For sites with a stability problem, the soil will initially have the same strength with or without wick drains installed. Therefore, in situations where stability is of concern, the rate of increase of load should be carefully controlled and monitored.

Wick drains can accelerate only primary consolidation. Therefore, it is important to estimate the magnitude and time rate of secondary consolidation. In these cases, means of minimizing the amount of secondary settlement by excess surcharge and/or extending waiting periods prior to final construction should be implemented. Soils with organic content should be carefully evaluated for their secondary consolidation characteristics.

Other limitations on the use of wick drains should be considered. Although they have been installed up to 60 meters in depth, the use of wick drains below 45 meters in depth should be evaluated by a specialist.

In most situations the flow properties of a good quality wick drain will not inhibit consolidation times. However, for extremely deep wick drains, combined with heavy loading and a relatively high in-situ soil permeability, the flow capacity of the system could be a limiting design consideration. In these rare cases, well resistance in the drain occurs and consolidation time will be determined more by discharge capacity of the drain rather than the horizontal permeability of the in-situ soils. Reference 1 and 5 contain specific technical guidance for this condition.

It is not recommended to install wick drains where the entire length or lower length requires predrilling. In these cases, sand drains may be considered a good alternative, or other methods of ground improvement.

2.5 ALTERNATE SOLUTIONS

Alternate solutions can be functionally divided into three different categories as follows:

1. Accept time constraints without the use of any vertical drain system.
2. By-pass the compressible soil, using deep structural foundations.
3. Reduce the compressibility of the in-situ soil.

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

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

- Use of a deep foundation is an effective and expensive means of by-passing compressible soils. This can be done by extending the bridge, or in the case of a structure, using a deep foundation. Such solutions are usually much more expensive and may require significant future maintenance. They also limit the flexibility of future uses of the site.

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

  - *Stone columns.* This method is used in soft subsurface soils to both accelerate settlement and provide sufficient strength increase to preclude deep seated global failure.
  - *Dynamic compaction.* Used mostly for compaction or consolidation of industrial fills and other near surface weak deposits.

- *Grouting.* Used to fill voids and in other specialized applications.
- *Deep soil mixing.* Used to change the in situ compression and strength characteristics of soils.
- *Excavation* and replacement of compressible soil.
- *Lightweight fills.* Used to reduce the embankment load.

All of these methods will strengthen the soil and reduce its compressibility, except lightweight fills. Where stability is a significant problem, combined solutions may be warranted in order to achieve the desired result. For example the most unstable areas can be treated with stone columns, grouting, or deep soil mixing and the remaining areas treated with wick drains, or excavation of weak in-situ soils and replacement with granular materials.

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

CHAPTER 3

CONSTRUCTION AND MATERIALS

3.1 CONSTRUCTION EQUIPMENT

There are many different ways of installing wick drains, but most employ the same principles. Table 2 lists the various methods of installation. With few exceptions, all methods in table 2 employ a steel covering mandrel that protects the wick drain material as it is installed. All methods employ some form of anchoring system to hold the drain in place while the mandrel is withdrawn. Another common feature is that the wick drain material comes in rolls and is threaded through the mandrel in a variety of ways. The major difference between the listed methods is in the technique used to insert the mandrel into the ground.

Table 2. Methods of wick drain installation.

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

* Presently used in the United States

Commonly used methods employ an installation mast that contains the material reels,, mandrel, and method of installation force. Added to this is a carrier, which is a crawler excavator or crawler crane, depending somewhat on the depth of installation.

A typical specification usually describes the acceptable method of installation(s) as static, vibratory, or static-vibratory. Other terms that have been used in lieu of static, are constant rate of advancement or constant load. These terms are really misnomers and do not fully describe the method of installation.

Figure 3 illustrates typical installation equipment and carrier pieces. The typical mandrel used in the United States has either a diamond shape of approximately 140 x 62 mm or a rectangular shape with dimensions approximately 50 x 130 mm. In some cases, especially with vibratory applications, a stabilizing fin will be necessary. In most installation masts, the wick drain material is fed through the mandrel from a storage reel mounted near the base of the mast. The material travels up the inside of the mast, over a sheave at the top, and down through the steel mandrel.

The sequence of installation operations starts when the steel mandrel has been threaded with wick drain material and attached to an anchor at the bottom of the mandrel. The anchor typically consists of a 13-mm-diameter piece of cable or rebar or a special anchor plate made of sheet metal. The rebar and cable anchors will be approximately 200 mm long and the sheet metal anchor plates will be made from a thickness sufficiently thin enough to allow the excess area to fold up around the steel mandrel to minimize its cross-sectional area to slightly in excess of the cross-sectional area of the steel mandrel. A small "handle" is spot welded to the sheet metal anchor for insertion of the wick drain material. When the sheet metal anchor is used, the wick drain material is inserted through the "handle" and reinserted in the bottom of the mandrel.

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

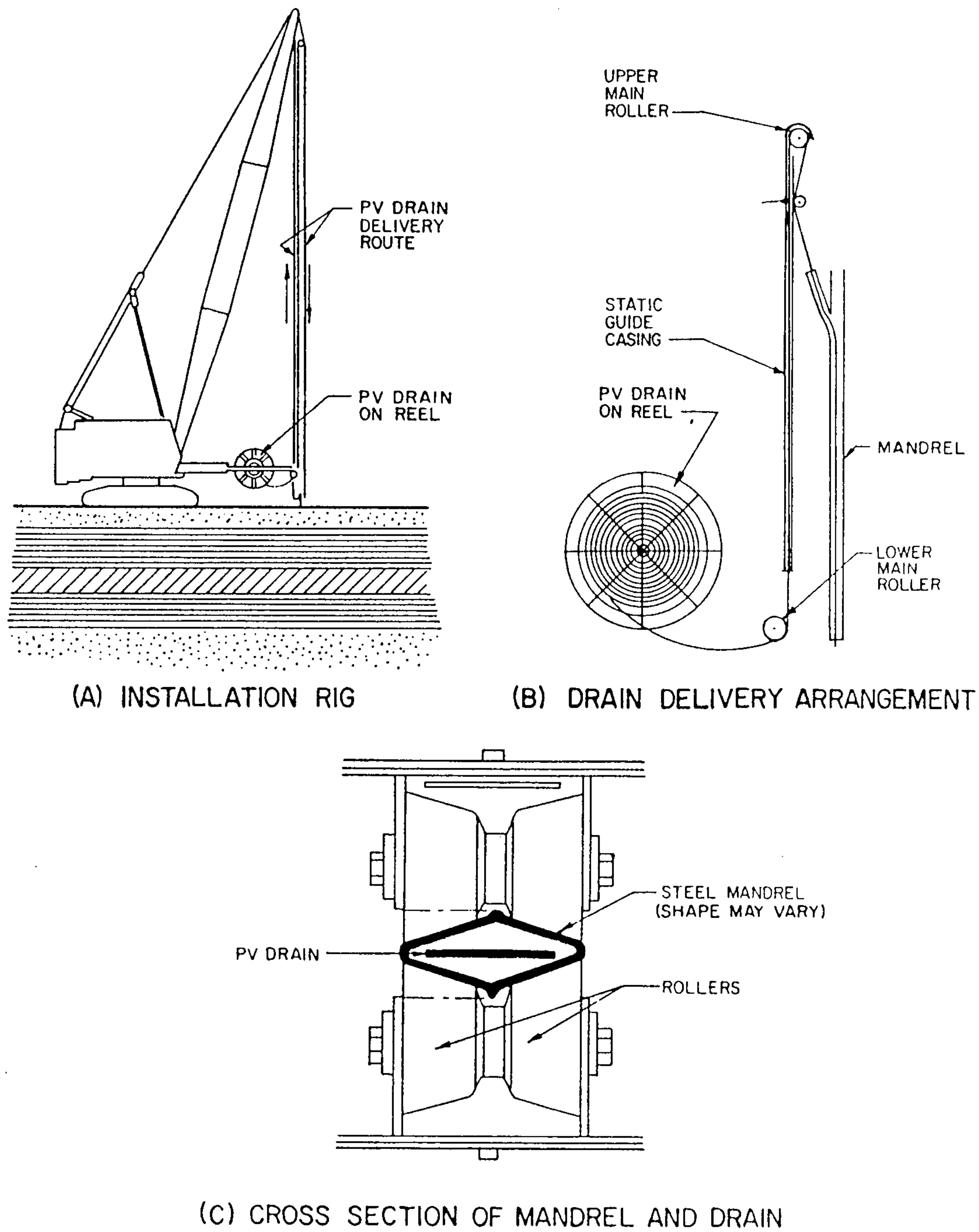


Figure 3. Typical wick drain installation equipment.

Except for unusual installation methods, where no mandrel is used at all, or where it is installed in sections, the length of the drain material has to be slightly longer than the desired installation depth. Therefore, extremely deep drains (over 45 m in depth) may require very large or specialized equipment. Wick drains have also been installed from large barges on water and with amphibious marsh buggies.

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

Once the wick drain material on the reel has been depleted, another is attached by splicing to the previous roll. The common method of splicing is to insert the end of the old roll into the new roll a minimum length of 150 mm. At this point at least 10 staples (4 on each side and 2 in the middle) will be used to hold the ends together. The splice is formed so that the bottom side of a vertical drain will be inserted into the upper end to ensure continuous flow.

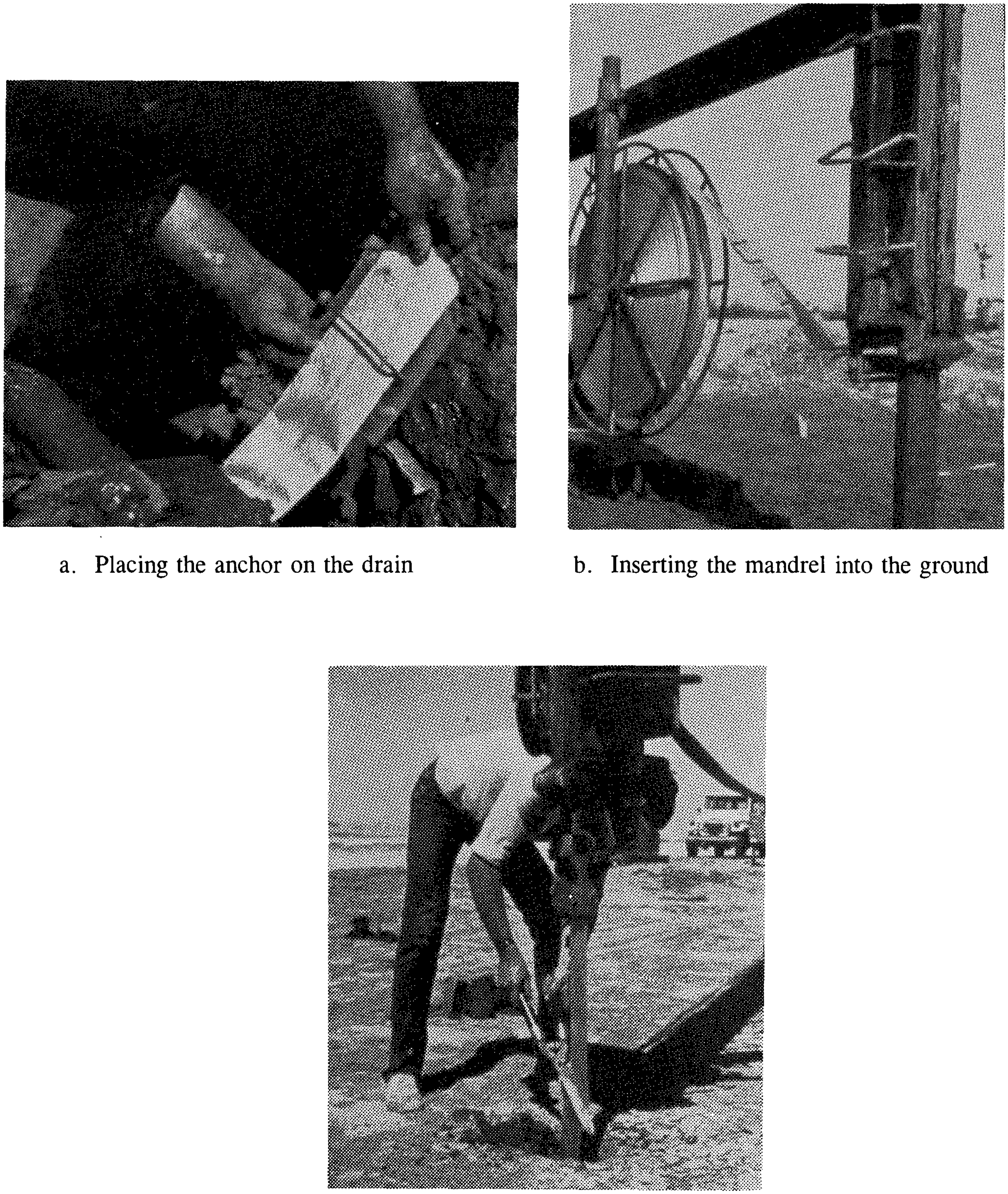
Normally, for wick drain installation, a one-to-two-person labor crew is required to handle the drain preparation, cutoffs, changing of drain rolls and anchor attachment. The only other labor required will be that to run the carrier piece and to perform occasional repairs to the equipment. Figure 4 and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

Specialized Equipment

While not commonly used, there are situations which will require very specialized equipment to install the wick drains. Situations where this might occur are:

- Unstable working surfaces
- Sloped surfaces
- Subsoils which are very difficult to penetrate
- Extremely deep drains (45 meters or more in depth)

Some examples of unusual equipment were mentioned previously in the discussion of normal installation systems. These include the need for special carrier pieces to do extremely deep drains and the use of marsh buggies on unstable working surface situations. Other examples include special external jetting techniques mounted on lightweight skid platforms used on steep slopes and the use of test boring equipment to install pipe drains with wick drain materials as relief wells in existing dams.

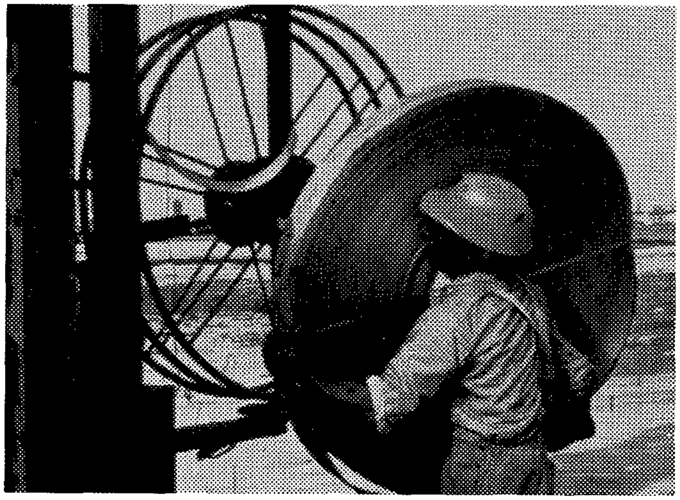


a. Placing the anchor on the drain

b. Inserting the mandrel into the ground

c. Cutting the drain after withdrawing the mandrel

Figure 4. Typical wick drain installation procedure. (Photographs provided by Geotechnics Holland, BV.)



a. Placing the new roll on the drail roller

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

c. Stapling the drain splice

Figure 5. Typical wick drain splicing procedure. (Photographs provided by Geotechnics Holland, BV.)

In one very unusual situation involving sludge deposits, the wick drains were installed from a row boat using a long plastic insertion rod. While these specialized methods are usually very costly on a per meter basis, they can be applicable especially where quantities are small.

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

3.2 MATERIALS OF CONSTRUCTION

a. Wick Drains

Wick drains are relatively flat and approximately 100 mm wide by 3-to-9 mm in thickness. The material generally consists of plastic core formed to make channels and a cover of a geotextile filter. These two components are equally important in the function of a wick drain.

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

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

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

b. Drainage Layer

The second component of a wick drain installation (or any vertical drain project) is the drainage layer or method of conveyance and discharge of water from the drains. In most cases, this is accomplished using a drainage blanket consisting of a granular material. Recently synthetic drains, often called strip drains, have been laid horizontally along the installation surface and connected to each individual wick drain. The purpose of the horizontal drainage layer is to provide a clear drainage path for the pore water to the atmosphere, without creating any head loss.

Where sand or gravel is used, the drainage blanket can also serve as a working platform to help support equipment used for wick drain installation. Ideally, sand should be clean and washed, such as concrete sand. The design of the drainage layer is based on its ability to transport the excess pore water without any head loss. If sand is used, it should have a minimum thickness of at least 300 mm, but more typically it is 0.5 to 1 meter. Where it is used directly over soft soils, typical designs require at least 300 mm of extra thickness because of potential contamination. Where there is a possibility of mud waving, as much as a 1-meter thickness has been used. If gravel is used, it should also be very clean. Because of its permeability, the gravel can be limited to a thickness of 200 mm with geotextiles on both sides keep it from being contaminated from intrusion from the lower soils or the upper embankment materials.

Where strip drain material is to be used in lieu of a drainage layer, there are several different techniques that can be utilized. Typically, the strip drain material is approximately 25 mm in thickness and either 150 mm or 300 mm in width. Depending on the flow rate of the individual strip drain and the length and number of drains attached to each strip drain, either the smaller or larger size may be necessary. In most cases, a strip drain is attached to every individual with drain. In other words, one horizontal strip drain for every row of wick drains is used. While in other cases a single strip drain has been used for as many as three different rows. In the latter case, each wick drain has to be connected to the strip drains. An example strip drain application is shown in figure 6.

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

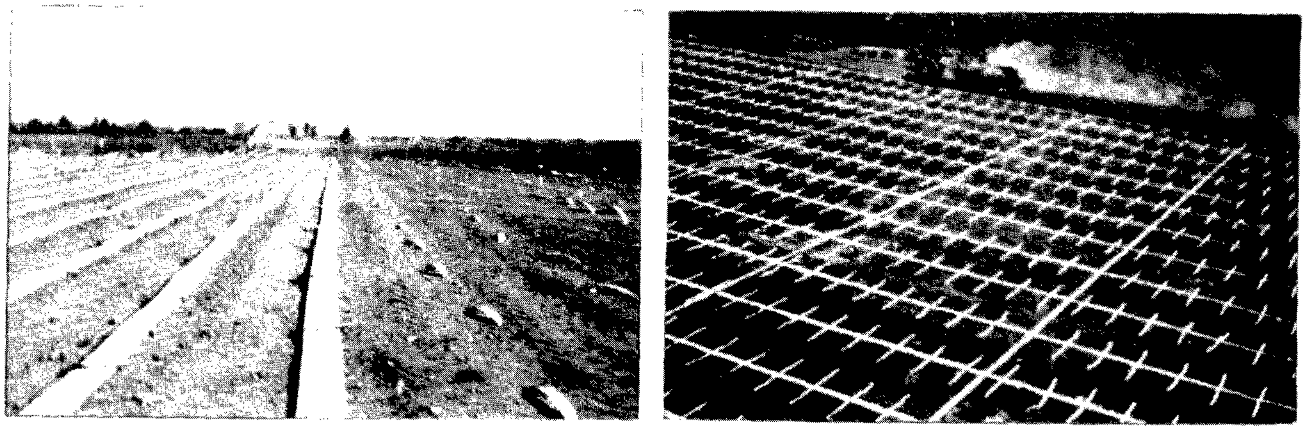


Figure 6. Strip drains.

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

CHAPTER 4

WICK DRAIN DESIGN CONCEPTS

The design concepts outlined in this chapter are based solely on the use of wick drains to accelerate consolidation. Therefore, the first objective of any design is to define the reason for the use of wick drains. For unusual applications such as reinforcement, liquefaction potential reduction and/or water table changes, specialized knowledge of the function of the wick drains will be necessary and are beyond the scope of this technical summary.

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

- Federal Highway Administration, "Prefabricated Vertical Drains - A Design and Construction Guidelines Manual." FHWA/RD-86/168, Washington, DC.

- Department of the Navy, Naval Facilities Engineering Command, "Soil Mechanics." NFAC Design Manual 7.1, May 1982, pp. 241-259.

The proper design of a wick drain installation requires knowledge of the type and extent of the foundation soils and their pertinent engineering properties. Engineering analyses must include, among other items, predictions of the amount and rate of settlement, both during and after construction and the embankment stability during all phases of construction. For consolidation analyses, the subsurface investigation program must define the extent and depth of the compressible strata and secure high quality undisturbed samples to determine past maximum pressures, coefficients of compressibility and coefficients of consolidation, both in a vertical and horizontal direction. In addition, Standard Penetration Test blow counts (N) are also significant in determining the installation costs and potential for the need for predrilling prior to placement of the wick drains.

4.1 DESIGN CONCEPTS

The principal purpose of any wick drain design is the selection of the type, spacing, and length of the drains to accomplish the required degree of consolidation within a specified time. This technical summary will concentrate on those issues.

The design process begins with the development of settlement analyses without wick drains to determine the total magnitude and the time rate settlement under the final embankment load. Wick drains may be a design solution if the time to reach 90 to 95 percent of these projected settlements is too great and beyond the contract construction time.

The assumptions used in developing one dimensional consolidation theory were applied to development of radial drainage theory related to vertical drains, which resulted in the following relationship between time, drain diameter, spacing, coefficient of consolidation and the average degree of desired consolidation.

$$t = \frac{D^2}{8\,c_h}\, F(n) m In \left[\frac{1}{1-\bar{U}_h}\right] \tag{1}$$

where:

t = time required to achieve desired degree of consolidation $\bar{U}_h$

$\bar{U}_h$ = average degree of consolidation due to horizontal drainage

D = diameter of the cylinder of influence of the drain (drain influence zone)

c_h = coefficient of consolidation for horizontal drainage

F(n) = drain spacing factor

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

d = diameter of an equivalent circular drain.

The above basic relationships can be modified to consider disturbance related to soil displacement during installation and well resistance. While these effects are often ignored for typical projects the reader should consult references 1 and 2, in order to fully evaluate and design for those effects.

Well resistance is rarely significant, except for extremely deep drains or where a combination of high loads and very permeable soils are present. For guidance in these situations, the applicable reference is 10, by Hansbo.

The following discussion of the components of equation 1 will give the reader a greater understanding of the factors that can effect the design when using wick drains.

- **c_h**. The horizontal coefficient of consolidation (c_h) of a particular layer can be obtained from laboratory consolidation tests. Even with proper laboratory techniques and high quality undisturbed samples, the designer is fortunate to be within 50 percent of the actual coefficient of consolidation. It is for this reason that designers may take a conservative or simplified approach in choosing a value for the coefficient of consolidation. Normally only the coefficient of vertical consolidation (c_v) is obtained from standard consolidation tests from which the horizontal coefficient is estimated.

  High quality laboratory tests have consistently shown c_h to be at least two times c_v, the coefficient of consolidation in the vertical direction. A common conservative approach is to assume c_h is directly related to c_v, without direct measurement values. For design c_h is generally taken as 1.5 to 2 c_v. Where lenses or layers of more permeable soils or varied conditions exist, a ratio of 2.0 to 3.0 is indicated and may be used.

  Equation 1 does not take into consideration any consolidation from vertical drainage. The vertical drainage effect may be minor when the compressible layer is over 5 meters in thickness and for a conservative approach, no beneficial effects may be considered for vertical drainage in preliminary feasibility evaluations.

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

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

- $\bar{\mathbf{U}}_{\mathbf{h}}$. Normally, an average degree of consolidation desired is 85 to 90 percent, unless very significant surcharges are designed. Fundamentally, it is a function of the magnitude of post construction settlement that the project can tolerate. It should be further understood that the contribution of vertical consolidation may be significant and should be considered for major/complex projects. Often it may approach 30 percent of the measured settlement.

- **D**. The diameter of the cylinder of influence of each wick drain. When using an equilateral triangular pattern, D is 1.05 times the spacing between each drain. In a square pattern, D is 1.13 times the spacing between drains.

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

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

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

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

4.2 FEASIBILITY DESIGN

While not necessarily applicable in every case, the designer can accomplish a simple preliminary determination of spacing a wick drain solution using soil data such as liquid limits and project geometry.

The definition of project geometry reflects the area of loading, depth of soft compressible strata, and whether excess surcharge will be necessary. An approximate coefficient of consolidation can be estimated from liquid limit values using the graph shown in figure 7.

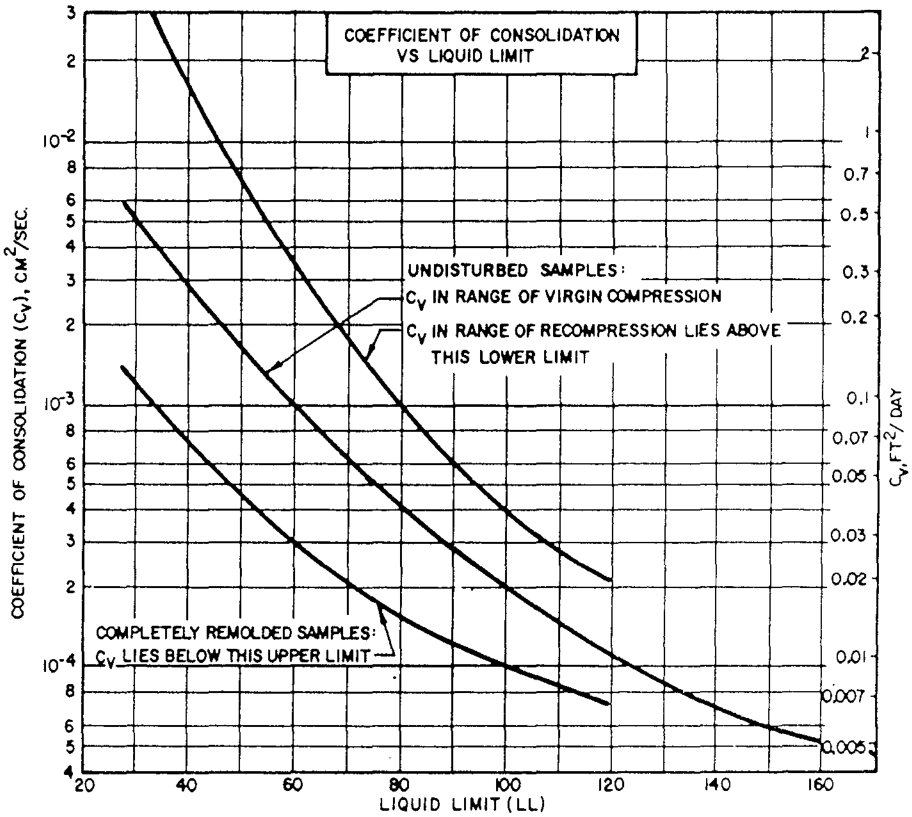


Figure 7. Coefficient of Consolidation vs. Liquid Limit.[8]

Assuming that the desired percentage of primary consolidation has been established or is in the 90 to 95 percent range, the designer can then either use computer programs available for vertical drain design, use the nomograph shown in figure 8, or solve equation 1 to determine a spacing consistent with available time.

If the nomograph is used, a determination of spacing for a 300-mm sand drain can be obtained. Theoretically, the wick drain spacing should be approximately 45 to 70 percent that of a 300-mm mandrel driven sand drain. Once the preliminary spacing is determined, the total project quantities can be computed from the project geometry.

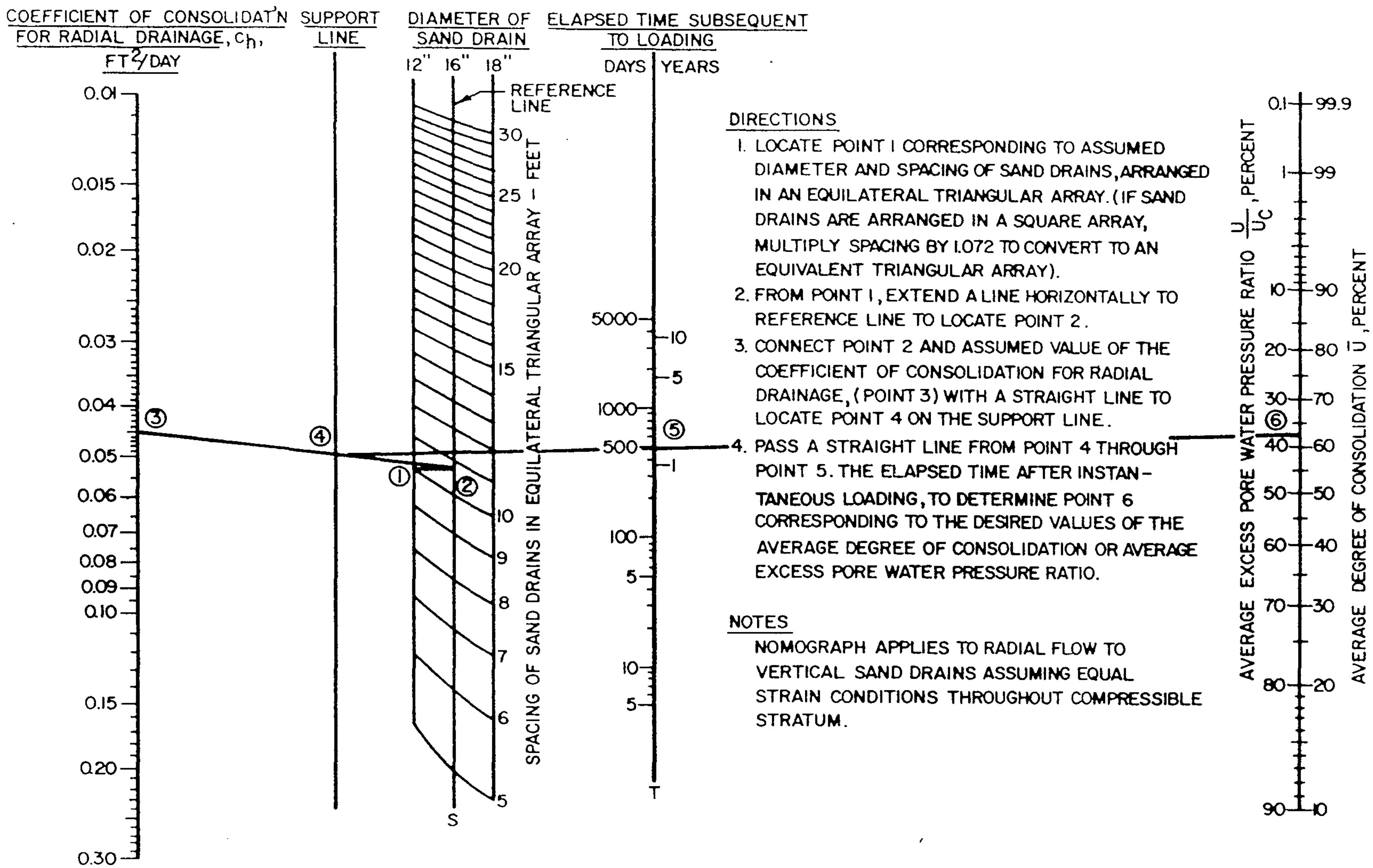


Figure 8. Nomograph for Consolidation with Radial Drainage to Vertical Sand Drain.[8]

4.3 FEASIBILITY EVALUATION EXAMPLE

A highway embankment is planned at a location where preliminary borings indicate a profile of 6 m of normally consolidated clay with sand lenses over rock. Total settlement, based on one-dimensional drainage is estimated at 300 mm, with 90 percent occurring in 10 years. This estimate is based on preliminary evaluations of compression and consolidation coefficients obtained from classification data, and liquid limit of 60.

Since the available construction time is on the order of 18 months, it is desirable to accelerate settlements by the use of wick drains to a point in which 90 percent of the total settlements would occur within about 1 year. To estimate feasibility and cost, a potential spacing must be determined.

From figure 4, a coefficient of consolidation (c_v) of 0.001 cm^2/sec is obtained and based on the existence of horizontal sand lenses in the profile, a coefficient of horizontal consolidation (c_h) of 2 times c_v can be assumed.

Equation 1 can be solved for various drain spacing, using an equivalent diameter of drain of 60 mm to obtain the field time required to attain 90 percent consolidation. The solution indicates required times on the order of 300 days at an approximate spacing of 2.5 m or 470 days for an approximate spacing of 3 m.

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

CHAPTER 5

CONSTRUCTION SPECIFICATIONS AND CONSTRUCTION CONTROL

5.1 SPECIFICATIONS

The role of good specifications is to reduce the problems, risks, and cost of wick drain installations and to ensure an installation that achieves the intended objective of the design. To accomplish the above, agencies usually specify the method and controls for wick drain of installation. End result specifications are not common since they would require extensive soil exploration and design by the contractor.

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

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

A general guideline specification follows. This specification is general in nature and should be modified for each project based on site conditions and performance requirements. A more detailed explanation on specifications, terms, and commentary is contained in Federal Highway Administration Report No. FHWA/RD-86-168.[1]

a. Description

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

b. Material

The wick drain shall be the prefabricated type compressed of a plastic core wrapped in a filter on non-woven geotextile material. The core shall be fabricated with suitable drainage channels.

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

1. Alidrain
2. Alidrain S
3. Aliwick
4. Amerdrain (Type 407)
5. Amerdrain (Type 417)
6. Mebra-Drain (7407
7. Mebra-Drain (MD 88)
8. (Castle Drain Board
9. Colbonddrain CX 1000
10. ....

c. Construction Details

Equipment

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

Installation Procedures

A minimum of four weeks prior to the installation of wick drains, the contractor shall submit details of the sequence and method of installation to the engineer for his review and approval.

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

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

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

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

Construction Requirements

Wick drains will be located, numbered, and staked out by the contractor. The location of the wick drains shall not vary by more than 150 mm from the locations indicated on the drawings or as directed by the engineer. The equipment shall be carefully checked for plumpness prior to advancing each wick, and must not deviate more than 50 mm per meter from the vertical. Wicks that are out of tolerance or that are damaged in construction, or that are improperly completed shall be rejected by the engineer, and no compensation will be allowed for any materials furnished or for any work performed on such wicks.

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

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

The contractor shall be permitted to use augering or other methods to loosen stiff or dense upper soils prior to the installation of the wick drains, provided that such augering or other method does not extend more than 600 mm into the underlying compressible soils. If preaugering is determined to be necessary, then the contractor shall be permitted to accomplish the preaugering prior to the installation of the drainage blanket. If the preaugering is performed prior to placement of the drainage blanket, a minimum length of 300 mm of wick drain material must extend above the working surface in order to provide continuous drainage into the sand blanket.

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

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

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

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

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

d. Method of Measurement

Wick drains will be measured by the linear meter for the full length of drainage wick complete and in place.

In case of obstructions, the contractor will be paid at the contract unit price for the number of linear meters measured from the theoretical top of the working platform to the elevation at which the obstruction was encountered.

e. Basis of Payment

Payment for wick drains will be made at the unit price per linear meter, which price shall be full compensation for the cost of furnishing the full length of wick drain material, installing the wick drain, altering of the equipment and methods of installation in order to produce the required end result in accordance with the plans and specifications and shall also include the cost of furnishing all tools, materials, labor, equipment, and all other costs necessary to complete the required work. No payment will be made for unacceptable wick drains or for any delays or expenses incurred through changes necessitated by improper or unacceptable material or equipment, but the costs of such shall be included in the unit price bid for this work.

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

Note: Where pre-augering of overburden is required, a separate pay item for the pre-augered length should be included.

5.2 INSPECTION GUIDELINES

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

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

This section outlines construction monitoring procedures that should be considered for any wick drain project. Other items of significance in construction such as the drainage blanket material quality, embankment placement and compaction, and surcharge loading rate must be monitored.

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

a. Site Preparation

Site preparation includes any excavation and grading to prepare the site for installation of wick drains. This may include site clearing, excavation and/or filling operations to bring the site to grade and construction of a working platform or drainage blanket. It is also important that the grade be such that a drainage blanket remains continuous and surface drainage will not erode the blanket to the extent where it is not longer continuous.

During construction of a working platform or drainage blanket the field inspection personnel should be monitoring for any unusual soil movements which would be indications of mud waving or a potential failure. On some projects, working mats or drainage blankets were installed too thick, creating failures which were very expensive to repair. In one project, a contract installed a 1.5-meter-high haul road over a 600-mm drainage blanket prior to wick drain installation in order to have access to a piling location. This haul road caused an extensive failure of the entire area that was to be stabilized.

b. Wick Drain Material and Installation Equipment

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

Material

Prior to installation, the wick drain material should be visually observed to ensure that it is similar to that originally submitted and tested, the core and filter jacket is continuous and complies to required dimensions.

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

Equipment

If a method specification is used, inspection personnel should determine that the equipment complies with specification requirements. Some of the important items to be checked are the following:

- Penetration method.
- Mandrel shape, size and stiffness.
- Anchor type and size.
- Method to measure and determine penetration depth.
- Method to measure and record installation force.
- Means and procedures for predrilling where necessary.

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

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

Submittals

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

The installation submittals should address the following:

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

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

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

5.3 MONITORING AND CONTROL

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

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

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

It is usually best to install settlement platforms and other instrumentation after the installation of the horizontal drainage layer and wick drains. This allows for an unimpeded site during wick drain installation and the ability to locate the most critical areas for instrumentation, especially for piezometers. If piezometers are installed prior to wick drain installation, it often becomes difficult to locate the drain at equal distance from piezometer location, especially where multiple piezometers are installed at different elevations. On some projects it might be valuable to install some instrumentation prior to wick drain installation. The resultant information may be useful for interpretation in critical situations. When stability is a major concern, slope indicators and some piezometers and settlement devices should be installed prior to wick drain installation. A typical layout with a complete set of instrumentation is shown on figure 1.

CHAPTER 6

COST EVALUATION

6.1 COST COMPONENTS

Often when reporting the cost of wick drains, only the actual cost of the installed wick drain is considered rather than the solution as a whole. Table 3 below details other factors affecting the total cost of the wick drain solution. Note that the unit cost of installation, whether including the mobilization or not, is only one many factors that effect the total cost of a wick drain solution.

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

• Project size, topography
• Obstructions, dense soils
• Allowable Construction and Consolidation Time • Allowable Postconstruction Settlement
• Preload and Surcharge - Type and Material Available - Reuse of Material - Amount of Surcharge
• Drainage Blanket or Horizontal Drainage Path
• Design, Instrumentation and Monitoring
• Unit Cost of Installation

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

- *Project topography:* The project topography can play a role in the cost of a wick drain solution. For example, it is very difficult, if not impossible, to install wick drains on significant slopes. If there is a great variation in grade on the site, significant earthwork may be necessary to provide a somewhat level working surface for installation.

- *Obstructions, Dense Soils.* The cost of wick drains increases significantly where soils are difficult to penetrate. Therefore, it is important to correctly identify the soils to be penetrated and the need to achieve specified depths. Depending on methods of installation, the designer should be concerned with very stiff layers (generally N values of 15 or greater); or, possible obstructions such as boulders, rocks, previous foundations, known underground utilities, etc. It is also difficult to anchor wick drains in extremely soft soils.

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

- *Consolidation Time.* Time required includes both consolidation time, time for installation of the wick drains, and the construction of the embankment and/or surcharge. Added factors effecting time, are the placement of instrumentation, site preparation, and drainage blanket installation. In general, the total time is the least variable factor because of project constraints.

- *Preload and Surcharge.* Preload is defined as the amount of fill material necessary to bring the site elevation to the final elevation. This includes the amount of additional embankment necessary to accommodate the final amount of settlement. Surcharge is the added fill above final elevations which is used to accelerate settlement or minimize secondary consolidation. The more surcharge that can be placed, the faster the consolidation.

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

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

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

6.2 SUMMARY OF TYPICAL PROJECTS

Typical cost of wick drain installations can be divided into three categories: small (3,000 to 10,000 m), medium (10,000 to 50,000 m), and large (50,000 m and greater).

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

Site Category	Unit Price Range
Small	\$2.25 to \$4.00 per m
Medium	\$1.60 to \$2.50 per m
Large	\$1.20 to \$2.00 per m

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

CHAPTER 7

CASE HISTORIES

The following case histories illustrate successful projects.

7.1 AIRPORT RUNWAY AND TAXIWAY EXTENSION

Project Description

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

Subsurface Conditions

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

Design Concerns

The designers were very concerned with post construction settlement resulting from the added fill, in order to maintain the proper grade for the runway. Settlement projections varying from 120 mm near the end of the existing runway to in excess of 300 mm at the greatest fill height could not be tolerated. Various solutions such as removal and replacement, surcharging, and wick drains were investigated. The high cost of removal and replacement eliminated this option. The time rate of settlement eliminated the option of just building the embankment and waiting for settlement to occur. Surcharging would not only be costly because of the need for removal and disposal after completion of settlement, but it would also create a slope stability problem.

The design also called for a twin set of 1.80 meter diameter culverts to be extended under both the runway and taxiway. While these culverts could be designed for the total amount of settlement anticipated, it was necessary that they be at the final elevations prior to opening of the runway.

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

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

Design

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

Project Results

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

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

Project Cost

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

7.2 APPROACH RAMPS FOR BRIDGE REPLACEMENT

Project Description

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

The new structure would be significantly shorter on the west end, meaning the new west approach embankment would extend 90 meters beyond the old abutment. The total alignment of the new embankment center line and bridge would be approximately 21 meters north of the existing center line. In addition, the existing bridge had to remain open until the new bridge could accommodate at least two lanes of traffic.

Subsurface Conditions

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

Design Concerns

Construction of the new approach embankments, varying in height from 5 to 11 meters, would result in considerable settlements. The west approach embankment would result in settlements ranging from 600 to 900 mm, without treatment. The settlement was expected to occur over a 6-year period after the full fill height was achieved.

On the east embankment, the proposed fill would incorporate much of the existing fills. Therefore, settlements of only 300 to 600 mm were expected, but the time of the settlement would be similar to the west approach. The designers used wick drains to accelerate the settlements to within a 6-month time frame.

On the west approach, a significant portion of the embankment south side slope would be under the existing bridge. It was felt that wick drains would be necessary in this area to prevent significant differential settlement of the side slope in future years. Therefore, some of the wick drains would have to be installed to maximum depths of 17 meters with a headroom clearance of only 8 meters.

On the east embankment there was no need for wick drains under the existing bridge, since the embankment did not extend past the old abutment. However, there was still need for wick drains adjacent to the existing embankment. Borings indicated that some wick drains in the new east embankment would have to penetrate slag under the existing embankment. The depth of the slag was 4.5 meters, and it would be difficult to penetrate, as indicated by very high "N" values in excess of 100 in some locations. The slag material was very abrasive and would be difficult to drill.

Analyses indicated that side slopes of 2H:1V could be safely constructed to a height of only 8.5 meters without foundation treatment. However, after the strength gain as a result of wick drain accelerated consolidation, embankments could be constructed to heights of 11 meters, as required.

Design

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

The total quantity of the original estimate was 70,000 linear meters of wick drains. Added to this quantity was the cost of 5,000 m^3 of a granular material for the drainage blanket and 16,000 m^2 of a geotextile. New York State DOT estimated the total cost of the wick drain solution, including auxiliary items, to be in an excess of $800,000.

Project Construction Procedure

Wick drain construction began on the east side and it became readily apparent that special drilling would be necessary to penetrate the slag fill on the area near the existing embankment. Special air rotary drills were used to predrill and the wick drain installation unit had to follow close behind. In areas farther from the existing embankment, a combination unit of vibro and static force was used to install the wick drains without predrilling. Approximately 25,000 m of wick drains were completed on the east side within 4 weeks.

Wick drains underneath the bridge on the west side were started simultaneously and completed in segments. A special sectional mandrel was developed such that it could be pinned together in a fairly rapid manner. The initial section was 6 meters long with subsequent sections of 5 m. At these locations, the holes had to be predrilled through a miscellaneous fill surface either with an auger or a special air track machine. After several days of trial procedures, a maximum production rate of 25 drains per day was achieved. This compared to a maximum production rate of over 300 drains per day on the locations not under the bridge.

The wick drains on the west side that were not under the bridge were completed in 4 weeks. It was also necessary to angle some drains under high tension wires in order to achieve full coverage of the area. The total quantity of wick drains installed on the west approach was approximately 33,500 m.

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

Project Results

Typical piezometer and settlement data from both the east side and west approach are shown in figures 9 and 10. Total maximum settlement of the west approach was 700 m, while on the east approach the maximum settlement was 600 mm. The settlement was achieved in both locations in approximately 4 to 5 months and agreed fairly well with predictions. While slope stability was not believed to be a concern, piezometers, slope indicators, and settlement plates were used to determine the time rate and amount of consolidation. The purpose for the slope indicators was two-fold. While stability was not though to be a concern, the factor of safety was marginal. Secondarily, it was believed that lateral movements could be used to determine the immediate settlement component.

Project Costs

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

The general contractor's bid prices were for (1) wick drains at $6.90 per m, (2) granular drainage blanket at $22.00 per cubic meter, and (3) geotextile at $1.70 per square meter. Using the original cost estimate, the actual prices resulted in a wick drain solution less than $600,000, which compared quite favorably to projected costs.

Subsequent to this project, New York State DOT has added a new bid item to account for wick drain mobilization costs. This bid item attempts to account for significant underruns in quantity and/or the need for special equipment for the projects.

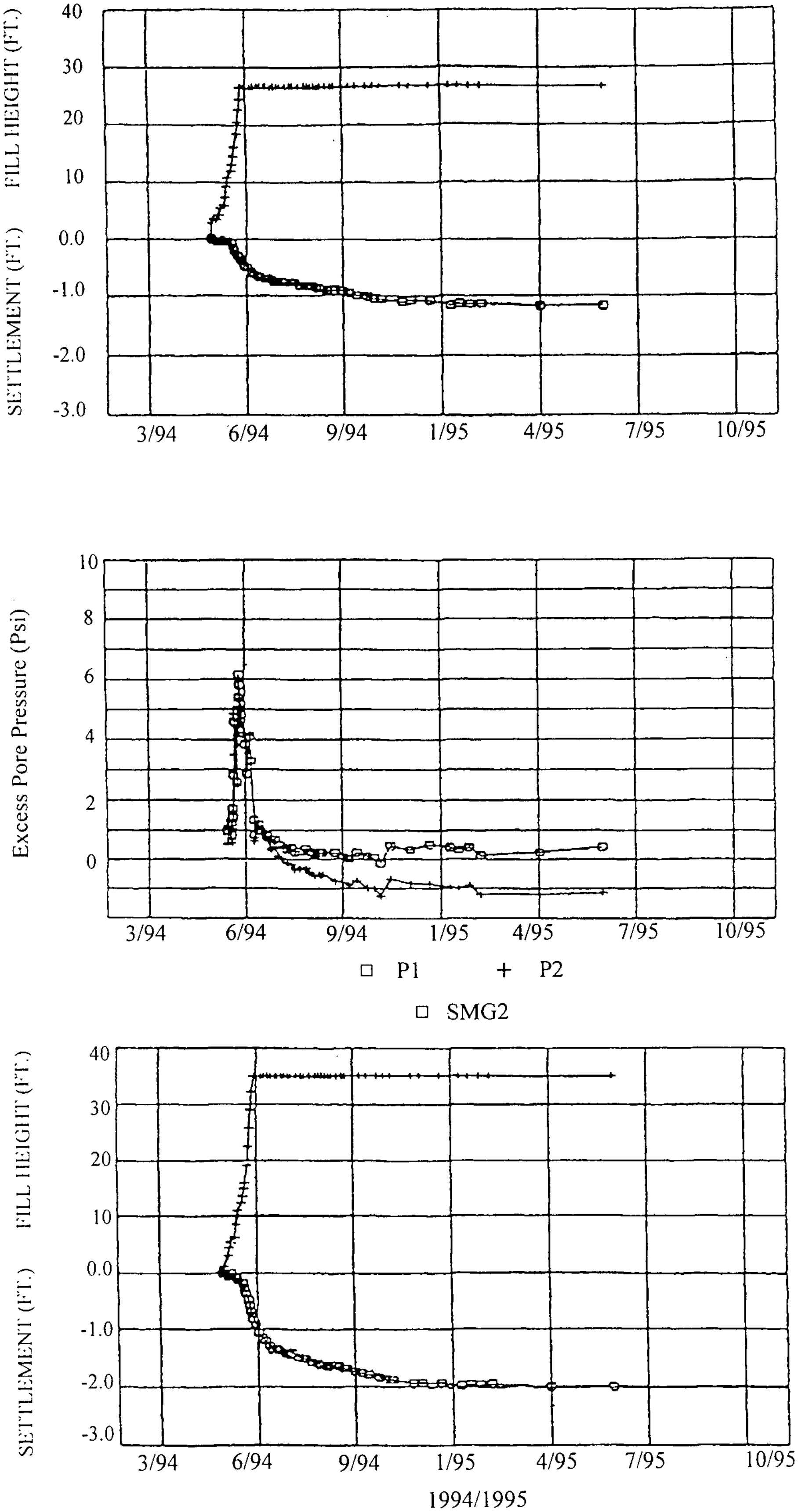


Figure 9. Tifft Street west approach embankment field data.

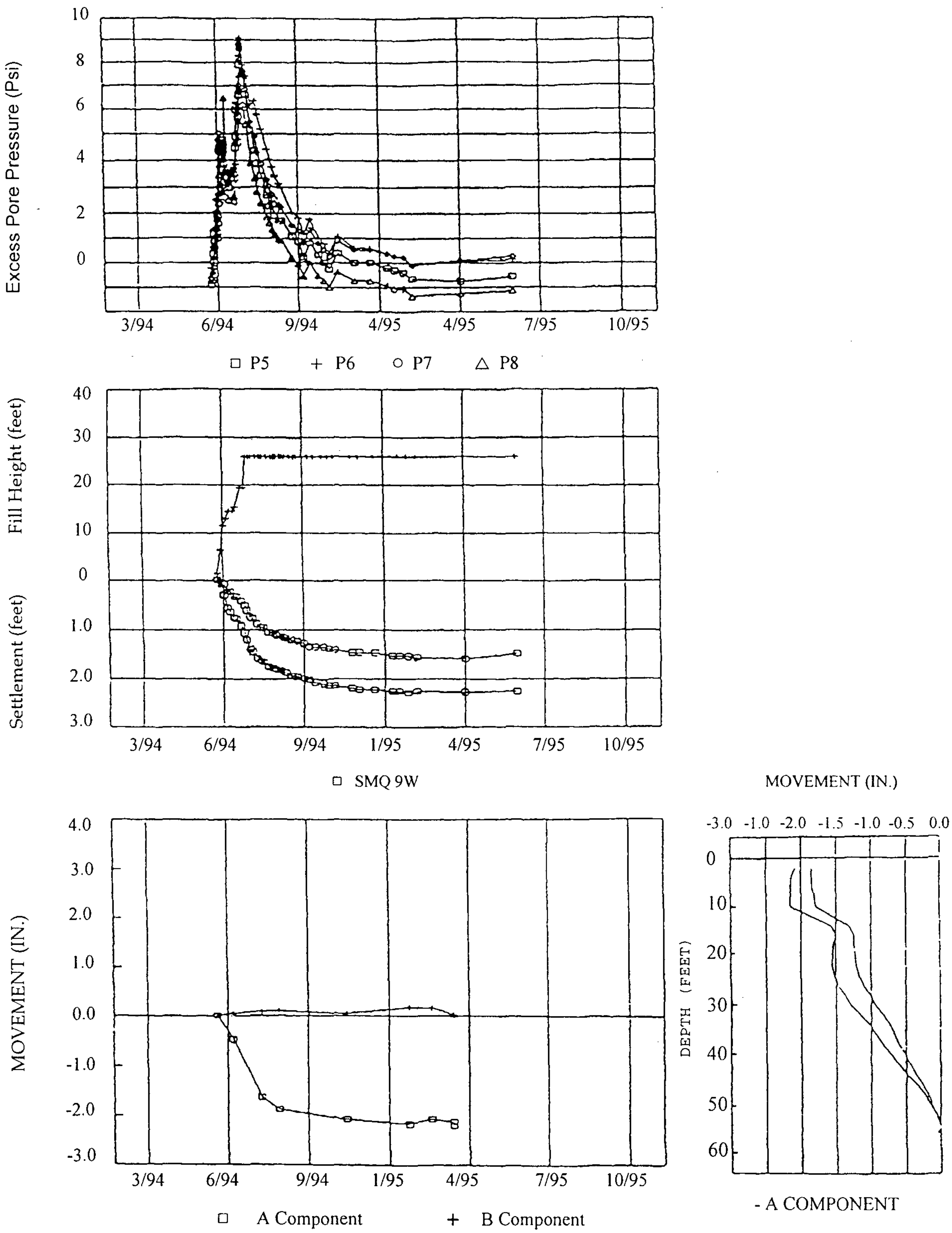


Figure 10. Tifft Street east approach embankment field data.

REFERENCES

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

2. _________ (1986), "Shared Experience in Geotechnical Engineering, Wick Drains," *Transportation Research Circular, Number 309.*

3. _________ (1990), "In Situ Soil Improvement Techniques," *AASHTO-AGC-ARTBA Joint Committee, Subcommittee on New Highway Materials, Task Force 27 Report.*

4. Holtz, R.D. (1987), "Preloading with Prefabricated Vertical Strip Drains." *Geotextiles and Geomembranes*, pp 109-131.

5. ________ (1994), "Calhoun County Rehab Uses Wicks to Drain, Speed Settlement," *Michigan Contractor & Builder, Clare Desmond, Vol. 88, No. 3.*

6. Barron, R.A. (1984), "Consolidation of Fine-Grained Soils by Drain Wells." *Trans. Am. Soc. Civ. Engrs. 113, Paper No. 2346.*

7. Charles, R. David (1984), "Performance of Vertical Wick Drains in Soft Soils." *Practical Application of Drainage in Geotechnical Engineering, Proceeding of the 15th Ohio River Valley Soils Seminar.*

8. Department of the Navy, Naval Facilities Engineering Command, (1982) "Soil Mechanics." *NAFAC Design Manual 7.1*, pp. 241-259.

9. Hannon, J.B. and Walsh, T.J. (1982), "Wick Drains, Membrane Reinforcement and Light Weight Fill for Embankment Construction at Dumbarton," *Transportation Research Record 897*, pp. 37-42.

10. Hansbo, S. (1979) "Consolidation of Clay by Band-Shaped Prefabricated Drains," *Ground Engineering, Vol. 12, No. 5*, pp. 16-25.

11. Koerner, Robert M. (1986), "Designing With Geosynthetics," *Prentice Hall*, Section 5.6, pp. 333-339.

12. Kyfor, Zenon G., Masi, Joseph J., and Gemme, Raymond L. (1986), "Performance of a Prefabricated Vertical Drain Installation Beneath an Embankment," Project: Hiawatha Blvd. I-81 Interchange. *New York State, Department of Transportation, Soil Mechanics Bureau.*

13. Morrison, A. (1982), "The Booming Business in Wick Drains," *Civil Engineering, Vol. 53, No. 3*, pp. 47-51.

14. Symposium in Print of Vertical Drains (1981), *Geotechnique, Vol. XXXI, No. 1.*

15. Haley & Aldrich, Inc. (1986), "Prefabricated Vertical Drains: Vol. 2, Summary of Research Effort," *FHWA/RD-86/169.*

16. Runesson, K., Hansbo, S., and Wiberg, N.E., 1985, "The Efficiency of Partially Penetrating Vertical Drains," *Geotechnique 35, No. 4*, pp. 511-516.

LIGHTWEIGHT FILL MATERIALS

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CHAPTER 1

DESCRIPTION AND HISTORY

1.1 HISTORICAL OVERVIEW

The compacted unit density of most soil deposits consisting of sands, silts, or clays ranges from about 1,800 to 2,200 kg/m^3. On some projects, it is desirable to use a material with a lower density in order to increase the factor of safety of the design. An example would be an embankment constructed on weak ground. The use of conventional earth fills could result in significant settlement or decreased stability such as a potential bearing capacity failure or a lower than desired factor of safety against a deep seated failure. In this situation, the use of a lightweight fill material would likely result in reduced settlement and increased stability.

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

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

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

Slags have been produced by the steel-making companies since the start of the iron and steel making industry. Initially, the slags were stockpiled as waste materials, but beginning around 1950, the slags were crushed, graded, and sold for fill materials.(2,3,4)

Geofoam is a generic term used to describe any foam material used in a geotechnical application. Geofoam includes expanded polystyrene (EPS), extruded polystyrene (XPS), and glassfoam (cellular glass). Geofoam was initially developed for insulation material to prevent frost from penetrating soils. The initial use for this purpose was in Scandinavia and North America in the early 1960s. In 1972 the use for geofoam was extended as a lightweight fill for a project in Norway. (1,5)

The technique of using pumping equipment to inject foaming agents into concrete was developed in the late 1930s. Little is known about the early uses of this product. However, the U. S. Army Corps of Engineers used foamed concrete as a tunnel lining and annular fill. This product is generally produced using regular material suppliers to provide the mixture of cement sand and water but then specialty companies provide the equipment and the foaming agents so that the foamed concrete is produced directly at the job site and pumped into position.[1,6,7]

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

Fill Type	Range in Density kg/m^3	Range in Specific Gravity	Approximate Cost[4] $\$/m^3$
Geofoam (EPS)	12 to 32	0.01 to .03	35.00 to 65.00[2]
Foamed Concrete	335 to 770	0.3 to 0.8	65.00 to 95.00[3]
Wood Fiber	550 to 960	0.6 to 1.0	12.00 to 20.00[1]
Shredded Tires	600 to 900	0.6 to 0.9	20.00 to 30.00[1]
Expanded Shale and Clay	600 to 1040	0.6 to 1.0	40.00 to 55.00[2]
Flyash	1120 to 1440	1.1 to 1.4	15.00 to 21.00[2]
Boiler Slag	1000 to 1750	1.0 to 1.8	3.00 to 4.00[2]
Air Cooled Slag	1100 to 1500	1.1 to 1.5	7.50 to 9.00[2]

[1] Price includes transportation cost
[2] FOB plant
[3] Mixed at job site using pumps to inject foaming agents into concrete grout mix
[4] See chapter 7 for details on cost data

Shredded tires are a relatively recent source of lightweight fill materials. The availability of this material is increasing each year and its use as a lightweight fill is further promoted by the need to dispose of tires. In most locations, the tires are stockpiled, but they are unsightly and present a serious fire and health hazard. Shredded tires have been used for lightweight fill in the United States and in other countries since the mid 1980s. Over 85 fills using shredded tires as a lightweight fill have been constructed in the United States.[1]

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

Expanded shale lightweight aggregate has been used for decades to produce aggregate for concrete and masonry units. Beginning in about 1980, lightweight aggregates have also been used for geotechnical purposes. Completed projects include the Port of Albany, New York marine terminal where lightweight fill was used behind a bulkhead to reduce the lateral pressures on the steel sheeting. Other projects include construction of roadways over soft ground. The existing high density soils were partially removed and replaced with lightweight aggregate to reduce settlement. Other projects have included improvement of slope stability by reduction of the gravitational driving force of the soil in the slope and replacement with a lightweight fill. [1,9,10,11]

Waste products from coal burning include flyash and boiler slag. Both of these materials have been used in roadway construction.[1] One of the first documented uses of flyash in an engineered highway embankment occurred in England in 1950. Trial embankments led to the acceptance of flyash fills, and other roadway projects were constructed in other European countries. In 1965, a flyash roadway embankment was constructed in Illinois. In 1984, a project survey found that flyash was used in the construction of 33 embankments and 31 area fills. Boiler slag has been used for backfill since the early 1970s. Many state highway department specifications allow the use of boiler slag as an acceptable fine or coarse aggregate.

1.2 FOCUS AND SCOPE

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

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

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

CHAPTER 2

APPLICATIONS AND LIMITATIONS

2.1 APPLICATIONS

Lightweight fill materials have many potential highway applications.

- All lightweight fill materials have a lower density than soil deposits. This is beneficial in reducing stresses imposed on underlying weak foundation soils.

- Most of the lightweight fill materials have favorable geotechnical engineering properties such as cohesion or an angle of shearing resistance that in turn is favorable in increasing the stability of an embankment against a slope-stability type of failure.

- Most of the lightweight fill materials have a compressibility similar to natural soils permitting pavements can be built either directly upon these deposits or with a buffer of soil between the pavements and the lightweight fills.

- Some of the lightweight fill materials have an inherently low Poisson's ratio which when combined with their low density is favorable in reducing lateral stresses that would otherwise be transmitted to retaining walls or tunnels.

- The granular lightweight fills are relatively pervious and drain rapidly, which is favorable for subgrade support.

- Some lightweight fill materials such as geofoams can be placed in wet or cold weather that would restrict or preclude conventional earthwork operations.

- The low density of lightweight fill materials is beneficial in seismic areas because the seismic inertial force is directly related to fill density.

Each of these items is further discussed in the following sections of this chapter.

a. Load Reduction

When an embankment is built on soft ground, the main driving force is from the weight of the embankment itself. Conventional methods of increasing the stability of the embankment have included:[12]

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

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

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

b. Shear Strength

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

Traffic loads introduce stresses into the subgrade soils. These induced stresses are resisted by the angle of shearing resistance or cohesion of the lightweight fill materials.

c. Compressibility

Many lightweight fill materials such as geofoam, foamed concrete, expanded shale and clay, flyash, boiler slag, and air cooled slag have a compressibility and elasticity similar to natural soils. Under static loading, the amount of settlement will be similar to that for conventional earth fill materials. Under dynamic loading, the resiliency of the subgrade will be similar to the natural soils.

For certain lightweight fill materials such as shredded tires or wood fibers, the compressibility of these materials under load is significantly larger than compacted soils, and this must be taken into account in the design. Furthermore, the resiliency of these materials is much larger than conventional soils. This requires a cover of at least 1 m of soil to be placed over the top of these deposits so as to reduce the resiliency, thereby enabling conventional pavements to be built on these deposits.

d. Lateral Pressures

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

e. Drainage Characteristics

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

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

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

f. Construction in Adverse Weather

It is difficult if not impossible to place and compact conventional soils during extreme cold weather. However, some lightweight fills such as geofoams have been successfully installed in bad weather. The first case history of chapter 6 describes one project where this was accomplished.

g. Seismic Considerations

In Japan, there have been case histories where a highway embankment constructed of geofoam did not fail in a severe earthquake even though adjacent sections of a soil embankment did.[13]

2.2 LIMITATIONS

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

- *Availability of the materials.* Certain geographic areas may have an abundance of one type of lightweight fill material but not of another. As an example, wood fiber fill would be available in lumber producing areas, flyash and slags in heavily industrialized areas, and expanded shales in areas where natural shale formations are present. Unless the lightweight fill material is available on a local basis, the transportation costs could raise the price considerably and make these materials non-competitive.

- *Construction methods.* Some lightweight fill materials could be difficult to place and handle. Fly ash behaves as a silt and when wet will be very spongy and when dry will dust easily. Shredded tires are very resilient when placed, thereby requiring somewhat unconventional compaction procedures. Foamed concrete requires the use of specialized equipment at the site to introduce air and other additives into the mixture before placement. This specialized equipment increases the cost.

- *Durability of the fill deposits.* Some lightweight fill materials must be protected to ensure longevity. Because geofoam is subject to deterioration from gasoline spills, geomembranes are generally placed over the surface of the blocks. Wood fibers can deteriorate over a long period of time, although recent studies indicate that the deterioration is limited to the outer surface of an embankment. Fly ash deposits need to be protected with a soil surface to minimize or prevent erosion of the side slopes.

- *Environmental concerns.* Some of the lightweight fill materials generate leachate as water passes through these deposits. Fortunately, design methods have been developed to minimize the amount of leachate, and, to date, these measures have worked satisfactorily. However, the additional cost of these measures should be considered during design.

- *Geothermal properties.* Most lightweight fill materials possess geothermal properties that are different than soil. This can lead to accelerated deterioration of flexible pavements and/or problems with differential icing of pavement surfaces due to an alteration of the heat balance at the earth's surface. Essentially, most lightweight fill materials act as thermal insulation even though this is not an intended or desirable function. However, this can be effectively controlled by placing a suitable thickness (500 mm, minimum) of soil and/or paving materials over the surface of the lightweight fill material.

CHAPTER 3

COMMON LIGHTWEIGHT FILL MATERIALS

3.1 INTRODUCTION

Lightweight fill materials include both recycled and manufactured materials. The recycled materials are generally the product left over from some industrial or commercial process. In the past, some of these materials were considered waste products and were landfilled or stockpiled. It has been found through experimentation that many of these materials have properties that are desirable for embankment construction such as low density, low unit cost, or relatively high permeability. The use of these materials in roadway embankments has increased during the past 20 years.

The manufactured materials consist of products that are specifically developed or manufactured for special design situations. These materials generally cost more than the recycled materials but have unique properties that satisfy certain needs.

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

A description of the common lightweight fill materials follows.

3.2 GRANULAR LIGHTWEIGHT FILLS

a. Wood Fiber

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

b. Blast Furnace Slag

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

- Air cooled slag. The molten slag is permitted to run into a pit. Solidification takes place under the prevailing atmospheric conditions after which cooling may be accelerated by water spray. After the pit has been filled and cooled, the slag is dug, crushed, and screened to the desired aggregate sizes. The resulting slag aggregate is angular and vesicular. The uniform graded slags have the lowest density, which is desirable for use as a lightweight fill material.

- Expanded slag. Treatment of the molten slag with controlled quantities of water accelerates the solidification and increases the cellular nature of the slag, producing a lightweight product. Either a machine or pit process may be used to mix the water and molten slag.

- Granulated slag. Molten slag that is quickly chilled will form a glassy granular product called granulated slag. This process is the most rapid of the cooling methods, and little or no crystallization occurs. The granulated slag may be crushed and screened or pulverized for various applications.

Air cooled blast furnace slag is the predominant form of slag processed in the United States, accounting for 90 percent of the blast furnace slag sales in 1992.(14) The expanded slags and granulated slag are more expensive to produce and their use is generally limited to aggregates for lightweight concrete or concrete block.

The principal constituents of blast furnace slag are the oxides of silica, alumina, lime, and magnesia. These oxides do not occur in a free form in the slag. Rather, they are combined to form various silicate and alumina silicate materials such as melilite. These oxides comprise 95 percent or more of the total. The remaining portion of the slag contains manganese, iron, and sulfur compounds.

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

c. Fly Ash

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

The chemical properties of fly ash depend upon the type of coal that is burned. ASTM standard C 618 defines two categories of fly ash. Class C fly ash is produced from lignite or sub-bituminous coal, and has pozzalanic and some cementitious properties. Some Class C fly ashes may contain lime contents higher than 10 percent.

Class F fly ash is produced from burning anthracite or some bituminous coals that meet the applicable requirements given in ASTM C 618. This class of fly ash has pozzalanic properties.

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

d. Boiler Slag

Boiler slag is produced in wet bottom and cyclone boilers. It has been estimated that approximately 25 percent of all power plant ash produced in the United States is boiler slag, 65 percent is fly ash, and 10 percent is dry bottom ash and cinders. Dry bottom ash is highly absorptive and not suitable for most embankment fills. However, boiler slag is a durable aggregate. It is formed when the slag flows from the furnace in a hot molten condition and is discharged in cold water where it crystallizes, solidifies, and forms angular black glassy particles usually less than 6 to 9 mm in size. It is generally composed of silica and ferric oxide particles of angular and irregular shape.

e. Expanded Clay or Shale

Synthetic aggregates can be produced from clays or shales by heating these materials in a rotary kiln to temperatures in the range of 1,000 to 1,200 degrees C for at least 15 minutes. In this process, the clay minerals of montmorillonite, illite, and kaolinite become completely dehydrated and will not rehydrate under atmospheric conditions. The expanded vitrified mass is then screened to produce the desired gradation for a particular application. The particles are subangular in shape. This aggregate has a hard ceramic outer shell with a porous core that contains air voids. The particles are durable, chemically inert, and insensitive to moisture.

The cost for producing expanded clay or shale materials is relatively high. For this reason, these products have generally been used as lightweight aggregates for structural concrete. However, in areas where high-quality, naturally occurring aggregates are no longer present, the expanded clays have been used to produce synthetic aggregates for normal roadway construction.

f. Shredded Tires

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

Since the tires are a waste product, the primary costs associated with the use of shredded tires are the shredding process and the transportation to the project site. Thus, the economics will depend upon the proximity of the highway project to a shredder.

3.3 LIGHTWEIGHT FILL MATERIALS WITH COMPRESSIVE STRENGTH

a. Geofoam

Geofoam is the generic name for any cellular (generally closed cell) material used in geotechnical applications. Most geofoam materials are plastic, but other materials such as glass have been used. The most commonly used geofoam material as a lightweight fill, is block molded expanded polystyrene (EPS) that is manufactured in a plant. The final product is generally a block similar to a large brick but with typical dimensions of 2 m long, 1 m wide and 0.5 m thick. Different size blocks can be made depending upon the size of the mold. Blocks can be factory of field cut to any size and shape.

The manufacturing process for EPS consists of expanding small beads of polystyrene into spheres that contain numerous closed cells. The expanded spheres are then fused into blocks in a heated vacuum chamber. The heat welds the expanded beads together to form a very light material with a high void content. The final product has an extremely low density but a relatively high strength and stiffness. It is possible to incorporate additives into the manufacturing process so that the blocks are flame retardant and insect infestation resistant.

b. Foamed Concrete

Foamed concrete is formed by introducing a foaming agent into the Portland cement matrix. Flyash is sometimes introduced, but without sand or gravel in the mix. The foaming agent produces interconnected air voids that result in a low density. The foaming agent is introduced at the job site using a special mixer and the foamed concrete is pumped as a fluid to the required location. Because of the high air content and the lack of coarse aggregates, the foamed concrete will flow in a manner similar to a viscous fluid.

After setting, compressive strength develops within the foamed concrete. The compressive strength is a function of the density of the in-place foamed concrete.

On actual projects, the material supplier will deliver the cement and water mixture, and then specialized firms that supply the foaming agents will blend these into the delivered cement and water matrix on the job site and pump it to the required location. The foamed concrete can be produced with only one experienced person and the mixing equipment at the site, using the cement and water mixture delivered by a regular concrete supply company and site laborers.

CHAPTER 4

DESIGN WITH LIGHTWEIGHT FILL MATERIALS

4.1 INTRODUCTION

When planning a project where lightweight fill materials might be appropriate, a number of factors need to be evaluated. These include:

- Availability of lightweight fill materials in the geographic area of the project.

- The geotechnical engineering properties of the lightweight fill material for use in the design evaluation. The same engineering analyses would be used in evaluating the suitability of the design using lightweight fill materials as for conventional fills. For example, if slope stability is a concern, the reduced density of the lightweight fill plus the angle of shearing resistance or cohesion of the lightweight fill should be included into the slope stability calculations. If settlement is a concern, the reduced density of the embankment should be used in the settlement calculations. References 12 and 15 discuss design procedures.

- An evaluation of the durability, water absorption potential, corrosion potential, and other unique characteristics of the lightweight fill materials.

- Design and construction considerations. It may be necessary to incorporate certain features into the design to compensate for potential environmental problems or for reducing erosion potential. For constructing with lightweight fill materials certain field procedures may need to be incorporated.

- An evaluation of the costs for using lightweight fill versus conventional construction. Costs for various types of lightweight fill projects are shown on table 1 and are discussed in chapter 7. However, the cost will vary for each particular site depending upon the availability of the lightweight fill materials to the project site.

4.2 DESIGN AND CONSTRUCTION GUIDELINES

As a guide for preliminary planning and design, tables 2 through 9 show some of the important geotechnical engineering parameters for lightweight fill materials. In addition, the tables also list environmental considerations and construction and design considerations. Each of these factors should be considered for designs using lightweight fill materials.

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

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

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

Considerable design information for geofoam blocks (EPS) has been recently summarized by Horvath in *"Geofoam Geosynthetic"*[5].

Table 2. Wood fiber design and construction guidelines.

Design Parameters	
Moist Density:	720 to 960 kg/m^3
Angle of Shearing Resistance:	
Sawdust	25° to 27°
Hogfuel	31°
Wood Chips	30° to 49°
Permeability:	1 x 10^{-5} m/s
Compressibility:	Loose volume reduces 40 percent on compaction Vertical subgrade reaction = 9 to 10 MPa in top 0.6 m roughly corresponding to a CBR of 1.

Environmental Considerations

Potential environmental effects of the leachate include:
- Depletion of available dissolved oxygen in ground water.
- Lowering of ground water pH cause of acidic nature of leachate which has pH of 4 to 6.
- Contamination of water with toxins.

Methods to reduce contamination include:
- Reduce water infiltration into wood fiber by drains and capping.
- Treatment of leachate.
- Barriers between wood fiber fill and adjacent bodies of water.

Design Considerations

Restrict particle size to 150 mm maximum to prevent development of large voids and less than 30 percent finer than 12 mm to minimize the use of fine uniform sawdust.

Use fresh wood fiber to prolong the life of the fill.

Use side slopes of 1.5H:1V or flatter.

Surface treatment with cover material of thickness 0.6 m or more to protect slope from erosion and to minimize deterioration of wood fibers.

Restrict height of fill to 5 m and reduce air penetration into wood to minimize the possibility of spontaneous combustion.

Construction Considerations

Truck-mounted equipment used to spread fiber in 0.3 to 0.5 m lifts.

Two coverages with a fully loaded hauling truck with a minimum mass of 15 Mg usually sufficient to properly compact wood fiber.

Table 3. Air cooled blast furnace slag design and consideration guidelines.

Design Parameters	
Compacted Moist Density:	1120 to 1500 kg/m^3-varies with size and gradation
Gradation:	Can be graded to any specified size from 100 mm and smaller
Angle of Shearing Resistance:	35 to 40°
Permeability and Compressibility:	Depends on final specified gradation Generally similar to gravel and sand
Environmental Considerations	
Slag contains small amounts of sulfur in combined alkaline compounds. The pH of water in contact with slag is generally in the range of 8 to 12, which tends to inhibit corrosion. Some washing of the aggregate may be required to control the pH to 11 or less to meet AASHTO specifications for pH of aggregates. There are no known environmental concerns. Slags have been placed below the water table and next to lakes and rivers.	
Design Considerations	
The slag behavior is similar to natural angular gravel and sand deposits. The highest internal stability occurs for aggregate that is well graded with a maximum particle size of 400 mm. The amount passing .074 mm should be limited to 5 to 7 percent. However, the density increases for well graded materials and if lightweight fill is desirable, the uniformly graded materials should be specified. Absorption in slags is usually in the range of 1 to 6 percent by weight. Slag is highly resistant to weathering and abrasion.	
Construction Considerations	
Slags can be placed and compacted in the same manner as natural gravel and sand.	

Table 4. Fly ash design and construction guidelines.

Design Parameters	
Density Range - Compacted	1120 to 1440 kg/m^3
Shear Strength:	33° to 40°, c = 0, for Type F Class C is self hardening so the shear strength will vary as it cures
Permeability:	Range of 1 x 10^{-6} to 1 x 10^{-9} m/s
Compressibility:	Cc = 0.05 to 0.37 Ccr = 0.006 to 0.04
Grain Size Range:	.005 to .074 mm
Specific Gravity:	1.9 to 2.5
Atterberg Limits:	Non-plastic
Environmental Considerations	
The leachate is alkaline with pH of 6.2 to 11.5. Calcium, sulfate, and boron are soluble constituents which can leach and migrate. The EPA has declared fly ash as non-hazardous.(16)	
Design Considerations	
Where the ground water table is high, a drainage blanket should be provided below the fly ash fill to promote a capillary cutoff to prevent frost heave and resiliency of the subgrade. Runoff from paved surfaces should be discharged into a drainage system. Surface waters from peripheral areas should be diverted away from the embankment to minimize infiltration into the flyash. The side slope of embankments should be covered with at least 0.6 m of soil to prevent erosion. If concrete is to be formed directly on fly ash, place a polyethylene barrier on the fly ash to prevent moisture absorption from the fresh concrete into the fly ash and to serve as a moisture barrier. Use fly ash in the concrete to reduce sulfate attack.	
Construction Considerations	
Fly ash behaves like silt: dusting will occur when dry and compaction is difficult when wet. Some means for adding water should be available on site to keep the water content near optimum for compaction. Surface protection to minimize erosion may be required. Compaction is obtained with smooth drum vibratory rollers or self propelled pneumatic tired rollers. Use 250 mm lifts and compact the fly ash immediately after spreading. The use of test strips to develop the most efficient compaction procedures is advisable.	

Table 5. Boiler slag design and construction guidelines.

Design Parameters Dry Density, Loose: 960 to 1250 kg/m^3 Dry Density, Compacted: 1440 to 1750 kg/m^3 Angle of Shearing Resistance: 38 to 42° Coefficient of Permeability: 0.3 to 0.9 mm/s Grain Size Range: .05 to 10 mm Atterberg Limits: Non-plastic Compressibility: Comparable to sand at same relative density
Environmental Considerations After 4 days of soaking, the pH of the water solution is generally in the range of 6.7 to 7.0. Barium has been detected by toxicity tests but at levels well below the RCRA specified standard. There are no known environmental concerns with the use of this material.
Design Considerations The aggregate is durable and satisfies acceptable limits for soundness tests. The aggregate works well as an underdrain filter material provided the gradation requirements are met. Side slopes should be covered with a minimum of 0.6 m of cover material since exposed material has low stability. Specify standard proctor compaction, AASHTO T-99, since some degradation occurs during laboratory compaction in accordance with AASHTO T-180.
Construction Considerations Compact with several passes of a pneumatic roller or a smooth-drum, vibratory roller. Keep water content at or above optimum water content as determined by AASHTO T-99. 6 to 10 passes are usually sufficient. Material must be kept wet since there could be a loss in stability when material dries.

Table 6. Expanded shales and clays design and construction guidelines.

Design Parameters

Dry Density, Compacted:	800 to 1040 kg/m^3
Dry Density, Loose:	640 to 860 kg/m^3
Angle of shearing resistance:	loose 35°, compacted 37 to 44°
Grain size gradation:	5 to 25 mm
Permeability:	High
Coefficient of subgrade reaction:	loose 9 to 10 MN/m^3 compacted 38 to 42 MN/m^3

Environmental Considerations:

There are no known environmental concerns.

Design Considerations:

The material will absorb some water after placement. Samples compacted at a water content of 8.5 percent have been found after 1 year to have a water content of 28 percent. Over a longer period of time, the estimated long term water content would be about 34 percent.

Buoyancy forces should be considered for submerged aggregate.

Side slopes of embankments should be covered with a minimum of 0.8 m of soil cover.

Use side slopes of 1.5H to 1V or flatter to confine the material and provide internal stability.

For calculating lateral earth pressures, use an angle of shearing resistance of 35°.

Construction Considerations:

Particle degradation can occur from steel-tracked construction equipment. Use 2 to 4 passes with rubber-tired rollers.

Fill should be unloaded at side of fill area and then distributed with lightweight equipment with a contact pressure of 30 kN/m^2 or less.

Optimum field density is achieved by 2 to 4 passes of rubber tire equipment. Use lift thickness of 1 m or less.

Field density may be approximated in the laboratory by conducting a one-point AASHTO T-272 density test.

Table 7. Shredded tires design and construction guidelines.

Design Parameters

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

Environmental Considerations

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

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

Combustion Potential

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

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

Table 7 (cont.)

Design Considerations Keep the shredded-tire fill above the water table. Limit layers to 3 m in thickness. Provide good surface drainage of roadway surface to avoid water seepage through the shredded tire fill. Use geotextiles above and below tire fill to prevent migration of surrounding soils into the fill. Limit maximum size of tire chip to 600 mm in length to prevent development of large voids. Metal fragments must be firmly attached to the chip and 98 percent embedded in the rubber to prevent exposed wire strands from puncturing tires or construction equipment. At least one sidewall must be severed from the face of the tire. A minimum 0.9 m thick soil cap should be placed on the top and side slopes of the tire chip fill to minimize pavement deflections and provide confinement.
Construction Considerations Spread using a track mounted dozer in a lift thickness of 0.9 m or less. Compact using sheepsfoot rollers, smooth drum rollers or by repeated passes with a D-8 dozer. Use multiple passes of compaction equipment since compressibility decreases after 5 to 8 cycles of loading. Anticipate 35 percent volume reduction during compaction plus 10 percent shrinkage under loading of soil cover and pavement base course.

Table 8. Expanded polystyrene (EPS) design and construction guidelines.

Design Parameters	
Density	12 to 32 kg/m^3
Compressive and Flexural Strength,	Varies with density, see table 10.
Modulus of Elasticity:	2.5 MPa to 11.5 MPa
California Bearing Ratio (CBR):	2 to 4
Coefficient of Lateral Earth Pressure:	Lateral pressures from adjacent soil mass may be reduced to a ratio of 0.1 of horizontal to vertical pressure.[1]
Environmental Considerations	
There are no known environmental concerns. No decay of the material occurs when placed in the ground.	
Design Considerations	
EPS blocks will absorb water when placed in the ground. Blocks placed below water have resulted in densities of 75 to 100 kg/m^3 after 10 years. Blocks above the water had densities of 30 to 50 kg/m^3 after 10 years. For settlement and stability analyses use the highest densities to account for water absorption. Buoyancy forces must be considered for blocks situated below the water table. Adequate cover should be provided to result in a minimum safety factor of 1.3 against uplift. Because petroleum products will dissolve geofoam, a geomembrane or a reinforced concrete slab is used to cover the blocks in roadways in case of accidental spills. For design, use a minimum compressive strength of 100 kPa. The maximum permanent stress should not exceed 30 percent of the compressive strength of the block at 5 percent strain. When considering line loads, the combined stress level should be within 2/3 of the compressive strength. Use side slopes no steeper than 2H:1V and a minimum cover thickness of 0.25 m. If a vertical face is needed, cover exposed face of blocks such as by shotcrete or other material to provide long term UV protection.	

Table 8 (cont.)

Construction Considerations The subsoil should be levelled before placement of geofoam blocks. A layer of sand/gravel is frequently placed as a leveling course. When multiple layers of geofoam blocks are placed, the blocks should be placed at right angles to avoid continuous vertical joints and to promote interlocking. Provide a mechanical connection between blocks using a barked plate for shear transfer. Place cover material over geofoam blocks as soon as possible to prevent displacement from wind or buoyancy.

Table 9. Foamed concrete design and construction guidelines.

Design Parameters	
Dry Density	335 to 770 kg/m^3
Compressive Strength:	Varies with density, see figure 1
Permeability:	10^{-5} to 10^{-8} m/sec depending on density
Modulus of Elasticity, E:	E = (dry unit weight) $^{1.5}$ (compressive strength) $^{.5}$
Coefficient of Lateral Earth Pressure:	Negligible for vertical loads applied directly over the foamed concrete. Lateral pressures from adjacent soil mass will be transmitted undiminished.
Environmental Considerations	
There are no known environmental concerns.	
Design Considerations	
Buoyancy could be a problem if foamed concrete is placed below the water table and there is not sufficient vertical confinement.	
The lower compressive strength mixes are affected by freeze-thaw cycles. The product should be used below the zone of freezing or a higher compressive strength used.	
There is some absorption of water into the voids, which could affect the density and compressive strength. Saturation by water should be prevented by construction of a drainage blanket and drains.	
Construction Considerations	
A staging area is required to mix the portland cement, water, and foaming agent on site.	
The foamed concrete is very fluid; formwork should be tight to avoid flow of the mix through joints or gaps in the forms.	
If the foamed concrete is placed in a confined area, forms are not necessary as the fluid mix will flow tightly to the restraining barrier.	
Lift thicknesses should be limited to 0.6 m to allow workers to shape the surface while wading in the fluid mix. If shaping is not required, the lift thickness should not exceed 1.2 m as the heat of hydration would have an adverse effect on the foam. Allow a minimum 12-hour waiting period between lifts. No special provisions for cold joints are necessary.	

Table 10. Typical physical properties of EPS geofoam.

Property	ASTM Test	Type XI	Type I	Type VIII	Type II	Type IX
Density kg/m^3 (lbs/ft^3) Nominal Minimum	C 303/D 1622	12 (0.75) 11 (0.70)	16 (1.00) 15 (0.90)	20 (1.50) 18 (1.35)	24 (1.50) 22 (1.35)	32 (2.00) 29 (1.80)
Thermal Resistance 25.4 mm (1.00 in) thickness minimum k•m^2/W (F•ft^{2}h/BTU) @ 4.4°C (40°F) @ 23.9°C (75°F)	C 177/C 518	0.58 (3.3) 0.55 (3.1)	0.70 (4.0) 0.63 (3.6)	0.74 (4.2) 0.68 (3.8)	0.77 (4.4) 0.70 (4.0)	0.81 (4.6) 0.74 (4.2)
Compressive Resistance at yield of 10% deformation Min. kPa (psi)	C 165/D 1621	35 (5.0)	69 (10)	90 (13)	104 (15)	173 (25)
Flexural Strength Min. kPa (psi)	C 203	70 (10)	173 (25)	208 (30)	276 (40)	345 (50)
Water Absorption by total immersion Max. Vol. %	C 272	4.0	4.0	3.0	3.0	2.0
Dimensional Stability (change in directions) Max. %	D 2126	2.0	2.0	2.0	2.0	2.0
Buoyancy Force kg/m^3 (lbs/ft^3)	-	961 (60)	961 (60)	961 (60)	961 (60)	961 (60)
Modulus of Elasticity (Young's Modulus) kPa (psi)	D 1621	3103 (450)	4655 (675)	5862 (850)	7935 (1150)	10344 (1500)
Stress kPa (psi) @ .5% Strain @ 1% Strain Poisson's Ratio	D 1621 -	17 (2.5) 35 (5.0) .05	24 (3.5) 48 (7.0) .05	29 (4.3) 58 (8.5) .05	41 (6) 82 (12) .05	55 (8) 110 (16) .05

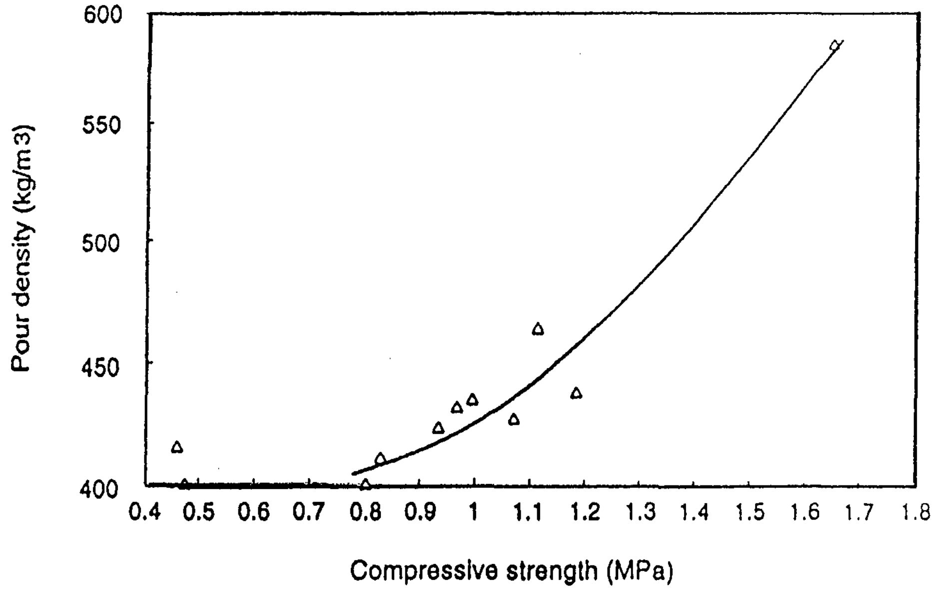


Figure 1. Pour density versus compressive strength for foamed concrete.(1)

CHAPTER 5

CONSTRUCTION SPECIFICATIONS AND CONTROL

5.1 SPECIFICATIONS

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

- The type of lightweight material that is to be used including the minimum and maximum density as delivered to the job site.

- The gradation of the fill material should be specified.

- Depending on the type of lightweight fill material used, other tests might be appropriate such as durability as measured by the Los Angeles abrasion test or the percent absorption of water.

- The lift thickness and the compaction procedure should be provided. In the case of lightweight fill materials that resemble normal soil sizes such as gravel, sand, and silt, the compaction can be specified in terms of a percent of AASHTO maximum density standards. For the case where recycled fill materials such as shredded tires or wood fibers are used, a method of compaction is more appropriate. This would include the type of equipment and the number of passes to be used to induce compaction. If there is any doubt as to lift thickness or percent compaction, consider test strips at the beginning of the field operations.

- A method of payment should be included in the specifications. A suggested procedure would be to base the payment on the amount of fill to be supplied in cubic meters on the basis of the planned final cross sections and the existing ground surface.

The lightweight fill that fall into this granular category includes wood fiber, blast furnace slag, flyash and boiler slag, expanded shales and clays, and shredded tires.

For lightweight fill materials such as EPS-block geofoam and foamed concrete that have inherent strength, the compressive strength is dependent upon how the product is constructed either in a manufacturing plant or at the job site by the addition of cement prior to deposition. For these lightweight fill materials, the specifications should include:

- The type of lightweight fill material to be used including the maximum and minimum densities that are tolerable.

- The minimum compressive and/or flexural strength.

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

Typical specifications for some lightweight fills are given in Appendix A.

5.2 MONITORING AND CONSTRUCTION CONTROL

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

In the case of shredded tires or wood fibers where compaction tests are not appropriate, field monitoring would include measurement of the lift thickness and the number of passes with the compaction equipment. Visual observations or measurements of the resiliency of the deposit during the multiple passes will also aid in determining whether additional passes of the compaction equipment are necessary. Test sections at the start of construction are desirable to either confirm the specified number of passes to make modifications at an early stage in the project.

For projects where the shredded tires will be used, field monitoring should also be undertaken to confirm that there is not excessive steel wire with the shredded tires or that materials other than rubber are being supplied. For projects where wood fibers are used, the gradation of the wood fiber should be checked on a daily basis to confirm that there is a blend of coarse sizes to the sawdust sizes. Only fresh wood fibers should be used to build the fills so as to prolong the life of the fill. The Washington State Department of Transportation has provided a classification for visually identifying different degrees of decomposition in the wood fiber.[17] Project personnel should be provided with this classification system so as to exclude the partially decomposed wood fibers from the new fill.

For the cohesive lightweight fill materials, field monitoring should include measurements of the density and compressive strength of the materials supplied. In the case of geofoams, the density and compressive strength will be developed in the manufacturing plant so samples of the delivered blocks can be obtained for these measurements. Observations of the placements of the blocks should also be made to confirm that the blocks are placed without a continuous joint and that shear transfer plates are installed between successive lifts of the blocks. The geomembrane covering the blocks should also be measured to confirm the thickness and that it entirely encloses the blocks. The seams within the geomembrane should be sealed properly.

In the case of the foamed concrete, the ingredients are mixed directly at the job site and then pumped to the location for use. Samples of the freshly mixed fill should be obtained in a manner similar to concrete testing for performance of density and compressive strength. The lift thickness of each pour should be measured to be sure that it does not exceed the maximum specifications and that there is sufficient hardening of one lift before the next lift is placed.

CHAPTER 6

CASE HISTORIES

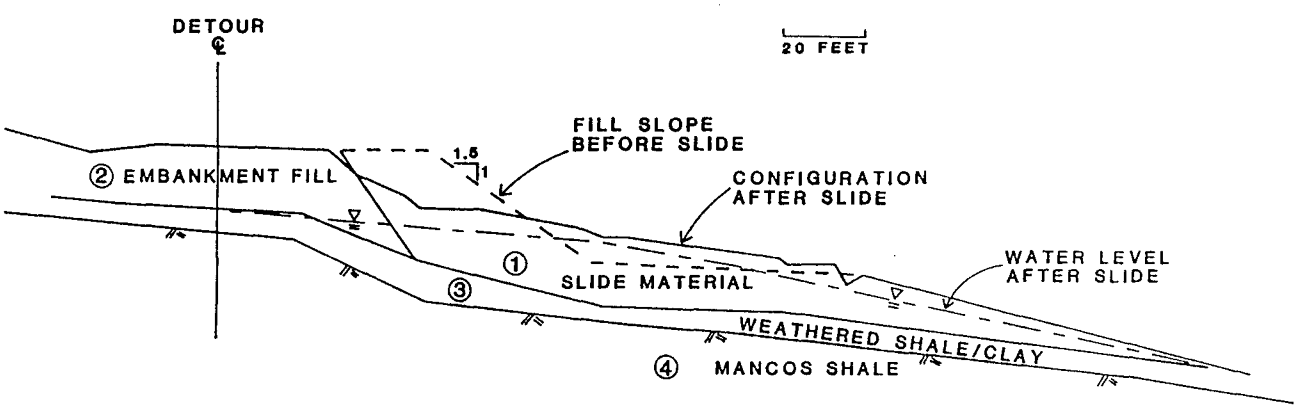
6.1 INTRODUCTION

Four case histories are presented to illustrate the various types of applications for which lightweight fill materials can be used in highway construction. These projects were selected because there was adequate information provided in the literature on the statement of the problem, the design considerations, construction methodology, and performance data.

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

6.2 Geofoam For Embankment Reconstruction

In 1987, a portion of U.S. Highway 160 in southwestern Colorado experienced a slide failure that closed the eastbound lane over a length of about 61 m.[18] Figure 2 illustrates the conditions before and after the slide. There was evidence of two older slides that were also present at the site of the problem but these slides were determined to be fairly stable.



SOIL/ROCK PROPERTIES

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

Figure 2. Geometry and cross section through slide area, Colorado project.[41]

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

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

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

Various alternatives were considered for restoration of the embankment. The first alternative involved removal of the entire slide debris and underlying weak soils and replacement with new fill. However, due to the presence of the old slides, right-of-way limitations, and the amount of earth involved, this option was deemed impractical.

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

The slide correction method consisted of two portions.

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

- The factor of safety was further increased by the use of EPS-block geofoam in the upper portion of the slide zone as indicated in figure 3. The material replacement reduced the driving forces. To prevent possible floating of the geofoam from ground water, a blanket drain was provided beneath the geofoam to conduct ground water away from this portion of the reconstructed area. A 250 mm perforated drain pipe was installed at the bottoms of the trench drains as shown in figure 3.

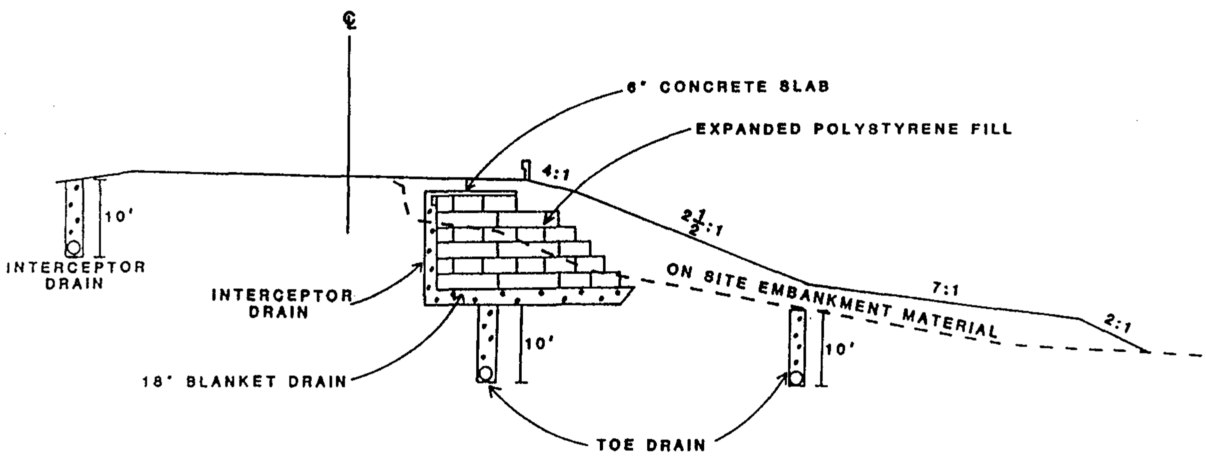


Figure 3. Designed embankment and drains - Colorado project.

Figure 4. EPS geofoam blocks being placed at Colorado site courtesy of BASF Corporation.

To monitor the performance of the embankment, settlement plates were installed in locations directly over the geofoam. Settlement readings indicate that there were 25 to 37 mm of compression of the geofoam as the soil overburden was placed on top of the geofoam during construction. Readings taken 1 year later indicate no additional movement although this was a dry year. The second year after construction was much wetter, and an additional 13 cm of movement was observed. There was also some lateral movement detected, but no particular pattern could be determined. Figure 4 is a photo showing the geofoam blocks being placed.

6.3 WOOD FIBER EMBANKMENT ACROSS SOFT GROUND

Approximately 180 m of two lane roadway was constructed over a swamp land in Washington State in 1987-88.[19] Figure 5 shows a generalized soil profile at the site including the existing and final roadway grades.

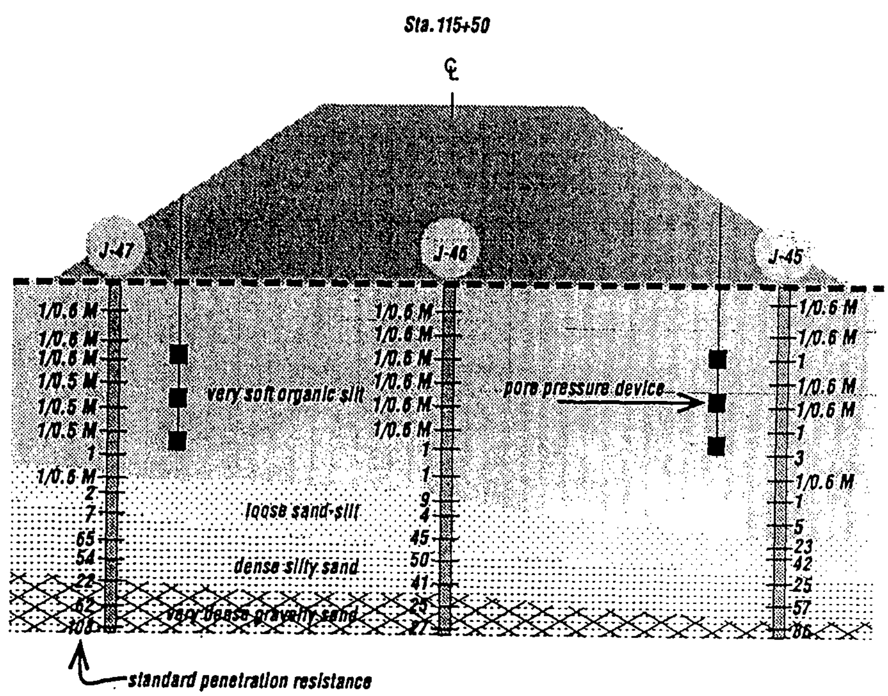


Figure 5. Soil cross section at wood fiber fill location Washington site.[21]

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

The new embankment vertical grades required up to 13.4 m of fill. Side slopes were limited to 2H:1V to limit the right of way and the fill volume required and to minimize the amount of wetlands removed by construction of the embankment. Slope stability calculations indicated that construction of the embankment using granular soil was not practical. By assuming no strength gain in the foundation soil, the maximum height of granular fill that could be safely constructed was determined to be 3.7 m. If the embankment were constructed slowly allowing pore pressure dissipation and strength gain, it was estimated that 20 months would be required for construction of the embankment. In addition, settlement under 13.4 m of granular fill was expected to be 2.4 m, which was unacceptable because of the necessity of installing a culvert at the base of the fill.

In order to obtain a minimum factor of safety of 1.25 for slope stability and 1.5 for bearing capacity, it was necessary to use a lightweight fill material for the embankment. Wood fiber was available in this area and was selected for the embankment fill. The initial 1.5 m of fill consisted of a silty gravelly sand to raise the base of the embankment above the prevailing ground water table. The upper 1.2 m of the fill was also a granular deposit to provide adequate subgrade support for the roadway. The remaining 10.7 m of the embankment height consisted of the wood fiber-fill. For the stability calculations, the density of the wood fiber-fill was used as 640 kg/m^3 and an angle of 40°. The granular soils were assumed to have a unit weight of 2 Mg/m^3 and an angle of internal friction of 37°. Five layers of geotextile reinforcement were embedded within the lower granular fill as well as within the wood fiber so as to meet the minimum factor of safety requirements for slope stability and lateral spreading considerations. The geotextile reinforcement was needed only until the soil gained sufficient strength to support the fill without reinforcement. It was estimated that this would require about 8 months and the geotextiles were designed allowing a relatively high creep limit of 60 percent of ultimate tensile strength.

Embankment settlement using the wood fiber fill was estimated to be 1.5 m plus an additional 0.13 m of secondary consolidation over a 20-year period. An accurate settlement estimate was necessary to determine the amount of additional wood fiber fill necessary to accommodate the anticipated settlement. Re-levelling with granular soils would reduce the slope stability factor of safety.

The as compacted density of the wood fiber fill was determined to be 610 kg/m^3. Compaction was obtained by routing hauling equipment over the entire lift thickness of 0.3 m. The minimum mass of the hauling equipment was 15 Mg.

Piezometers were installed in the organic silts and monitored as the embankment was raised in height. The maximum allowable pore pressure ratio was equal or exceeded twice during construction. This occurred when the fill height reached 6.6 and 9.5 m. In the first case, fill construction was stopped for 52 days and in the second case for 130 days.

The total time required to construct the embankment was just under 11 months. The subgrade was reached in September 1987 and paving began in October 1988. The measured settlement of the embankment immediately following construction to subgrade level was 1 m. Just prior to paving, this settlement had increased to 1.2 m. In September, 1992 settlement increased to 1.4 m. This compares favorably with the estimated primary settlement of 1.5 m.

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

6.4 WEIGHT COMPENSATION WITH FOAMED CONCRETE

The New York Department of Transportation has used foamed concrete fill on 12 separate projects. The typical application was for balanced excavations.[(7)] In this procedure, the existing soils are excavated to a depth required to balance the weight of the lightweight foamed concrete fill which would be placed to a grade higher than existing. Thus, no additional load was applied to the foundation soils. Generally, 1 m of foamed concrete fill at a density of 640 kg/m^3 will have the same weight as 0.35 m of existing fill with a density of 1920 kg/m^3.

A typical example of weight balancing was for a two-span structure that was replaced with a single span structure. The abutments were structurally sound and were used to support the new superstructure. It was necessary to raise the grade of the approach embankments by 1 m. This is not a large grade increase, but the embankments were underlain by 9 to 10 m of very soft to soft clay which in turn was underlain by loose silt eventually grading to a very compact silt. An analysis of the bearing capacity and slope stability of these soils indicated that the raising of the grade could jeopardize the factor of safety.

To reduce the loading on the soft soils, the upper 2 m below final grade was constructed of foamed concrete with a maximum unit weight of 480 kg/m^3. A drain was placed behind the existing abutment so as to reduce water pressures on the abutment. Figure 6 illustrates the final design and figure 7 placement of the foamed concrete.

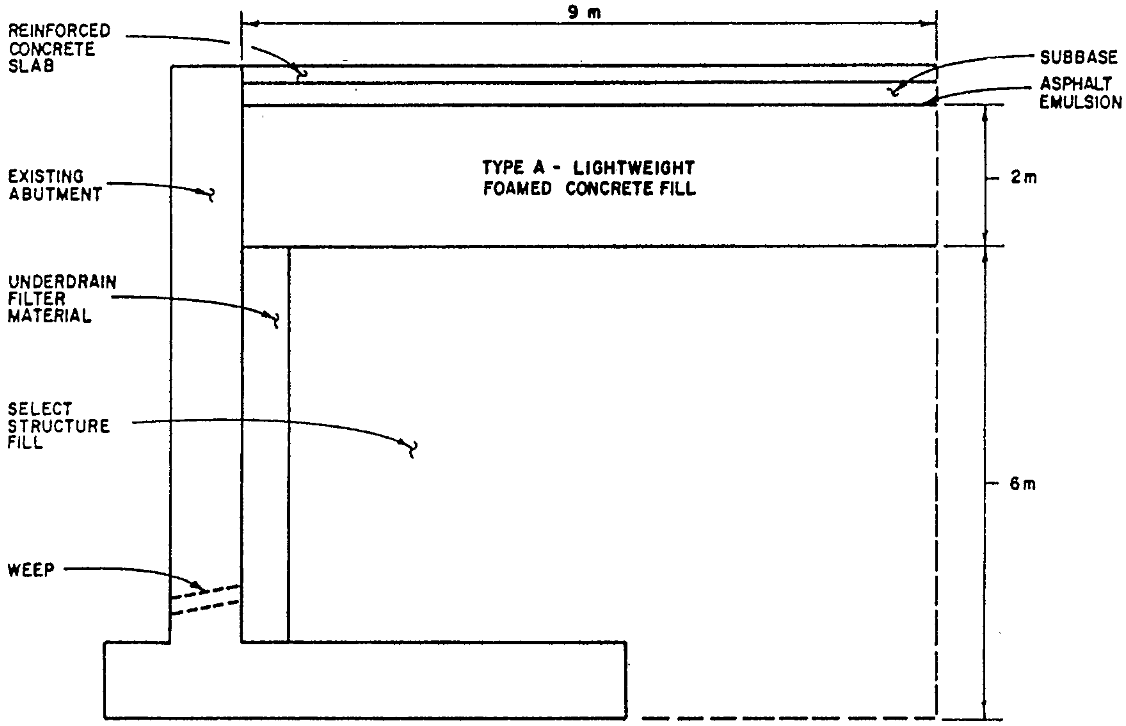


Figure 6. Foamed concrete fill behind existing abutment New York project.[14]

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

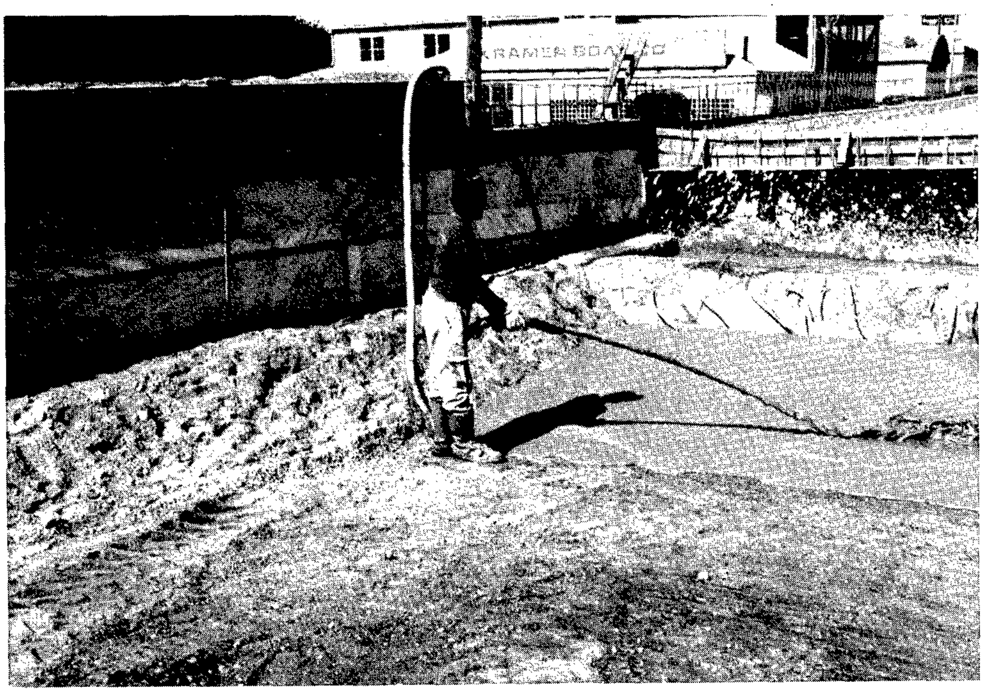
- The foamed concrete should generally be placed in 0.3 m maximum lifts. To establish a side or end slope, the fill can be placed in step lifts and then topped with conventional fill.

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

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

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

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



Courtesy of Aning Johnson Co.

Figure 7. Foamed concrete being placed for embankment in Illinois.

6.5 Shredded Tires For Repair Of A Landslide

An existing highway embankment in southern Oregon was widened 6.1 m and raised 1.2 m.[20] The additional embankment load remobilized an old landslide that moved progressively down slope perpendicular to the highway. The geotechnical investigation indicated that the slide movement could be arrested by reducing the embankment load and adding a down-slope counterbalance. The material for the counterbalance came from the soils that were excavated from the existing embankment. Various lightweight materials were considered for reconstructing the embankment, but shredded tires were selected because of the favorable material costs, including a State rebate for beneficial use. The typical cross section of the repair scheme is shown on figure 8. A rock drainage blanket was used beneath the shredded tire fill to isolate the tires from the ground water table. This was done for environmental protection purposes. The tire chips were positioned using dump trucks to place the fill at the end of a prepared area. A D8 dozer was then used to spread and compact the tire chips in 0.9 m lifts. The dozer was routed back and forth first in a longitudinal direction and then in a transverse direction; this comprised 1 coverage. At least 3 coverages were completed for each lift.

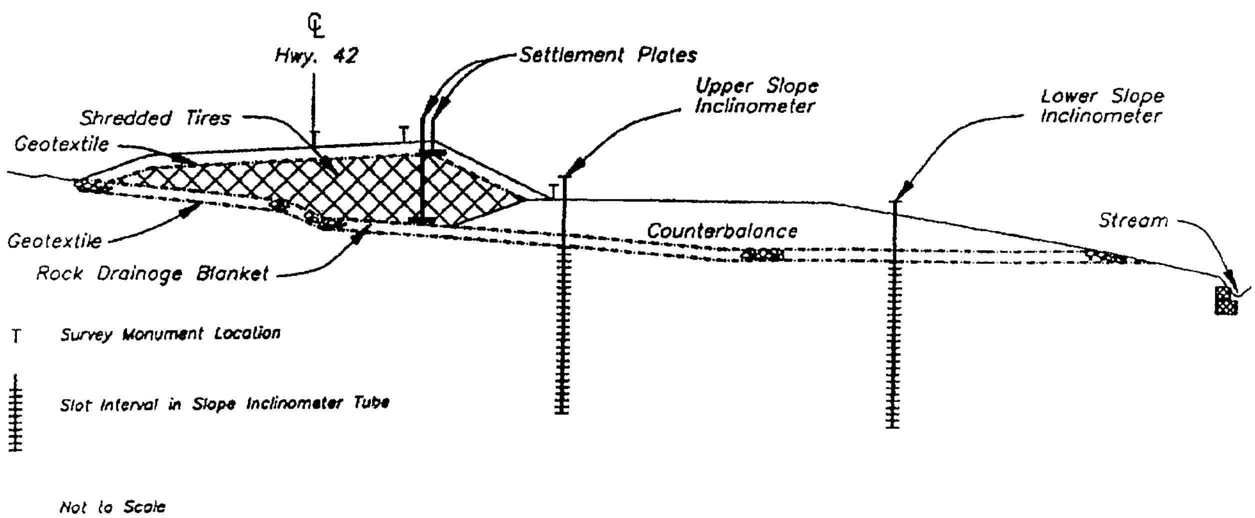


Figure 8. Typical cross section, shredded tire fill site in Washington.[38]

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

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

Settlement plates were installed within the embankment with one plate at the bottom of the shredded-tire layer and one at the top. This allowed for calculation of the density of the shredded tires in several different conditions including the density as compacted by the dozer, after surcharging by the soil cap and pavement section, and finally after 1 year of traffic. The densities calculated from these settlement surveys plus measurements taken on the trucks as hauled to the site are summarized in table 11.

Table 11. Shredded tire densities.[20]

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

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

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

CHAPTER 7

COST DATA

Costs for lightweight fill materials will depend upon a number of factors including:

- The basic cost for the material itself. If the material is a waste product that can be used without additional recycling, such as wood fiber, the cost will be relatively low. Recycled materials are also relatively cheap, but crushing or shredding will increase the cost slightly.

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

- Availability of materials. If the materials are produced in very low quantities, there may not be sufficient materials available unless multiple sources are used for the product.

- Placement or compaction costs could be higher than for soil fill. This would include moisture control for flyash and boiler slag, geomembranes for geofoam, or greater numbers of passes for shredded tires.

When calculating the costs for most lightweight fills, a common denominator could be used. Generally, prices for granular materials are quoted by suppliers as a cost per ton of material. However, the density of lightweight materials varies considerably. A ton of lightweight fill material will provide a much greater volume than conventional soils with a higher density. For this reason, the cost comparison should be made on the price per cubic meter. The conversion from dollars per ton to dollars per cubic meter can be made with the following simplified expression:

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

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

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

Typical unit prices that are reported in the literature for various types of lightweight fill materials are summarized in table 12. In some cases, available cost information is limited, and the prices could vary significantly from that shown in the table. These prices were generated for projects constructed during 1993 to 1994, and could vary due to inflation as well as other variables. The source of the cost is also included in table 12.

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

Table 12. Typical costs for lightweight fill materials.

Lightweight Fill	Cost Per Cubic Meter	Source of Costs
Wood fiber	\$12.00 to \$20.00[1]	Reference 4
Air cooled blast furnace slag	\$7.50 to \$9.00[2]	S
Expanded blast furnace slag	\$15.00 to \$20.00[2]	S
Boiler slag	\$3.00 to \$4.00[2]	S
Fly ash	\$15.00 to \$21.00[2]	S
Geofoam blocks (EPS)	\$35.00 to \$65.00[2]	S
Foamed concrete	\$65.00 to \$95.00[3]	S, Reference 7
Expanded shales and clays	\$40.00 to \$55.00[2]	S, Reference 7
Shredded tires	\$20.00 to \$30.00[1]	Reference 20

NOTE: These prices were obtained for projects completed in 1993-1994. Current costs could vary due to inflation.

(1) Price included transportation cost
(2) FOB plant
(3) Mixed at job site
S Supplier of lightweight fill material

REFERENCES

1. __________ (1997) "Lightweight Filling Materials," (1997) Technical Committee 12, *Permanent International Association of Road Congrèsses*, AIPRC, La Defense Cedex 04, France.

2. Lewis, D. (1982), "Properties and Uses of Iron and Steel Slags."

3. National Slag Association (1968), "Building Safer Highways with Slag," *American Road Builders Association, Technical Bulletin No. 264.*

4. National Slag Association (1988), "Processed Blast Furnace Slag, The All-Purpose Construction Aggregate," *NSA Bulletin 188-1.*

5. Horvath, J. (1995), "Geofoam Geosynthetic" *Horvath Engineering P.C.*, Scarsdale, NY.

6. __________, (July 22, 1976), "Foamed Concrete in Lightweight Fill," *Engineering News Record.*

7. Harbuck, D.I. (1993), "Lightweight Foamed Concrete Fill," *Transportation Research Record No. 1422*, Transportation Research Board, pp.21-35.

8. __________ "Interim Guidelines for Shredded Tire Embankments," Director Office of Engineering, HNG-23, Washington DC.

9. Holm, T., Valsangkar, A. (1993), "Lightweight Aggregate Soil Mechanics: Properties and Applications," *Transportation Research Record No. 1422*, Transportation Research Board, pp.7-13.

10. Stoll, R., Holm, R. (1985), "Expanded Shale Lightweight Fill: Geotechnical Properties," *Journal of Geotechnical Engineering, ASCE, Vol III, GT8*, pp.1023-1027.

11. Valsangkar, A.J., Holm, T.A. (1990), "Geotechnical Properties of Expanded Shale Lightweight Aggregate," *ASTM Geotechnical Testing Journal, Vol. 13*, pp.10-15.

12. Holtz, R. (1989), "Treatment of Problem Foundations for Highway Embankments," *NCHRP Synthesis of Highway Practice 147*, Transportation Research Board.

13. Expanded Polystyrol Development Organization (1994), "Slope Stabilization by the EPS Method and Its Applications," *International Geotechnical Symposium on Polystyrene Foam in Below Grade Applications*, Honolulu.

14. Solomon, C. (1993), "Slag-Iron and Steel," *U.S. Department of Interior, Bureau of Mines Annual Report*.

15. __________ (1975), "Treatment of Soft Foundations for Embankments," *Synthesis of Highway Practice 29*, Transportation Research Board.

16. Rittenhouse, R.C. (October 1993), "EPA, Miracle Concrete, Non-Hazardous Wastes," *Power Engineering Magazine*, p.14.

17. Kilian, A.P., Ferry, C.D. (1993), "Long Term Performance of Wood Fiber Fills," *Transportation Research Record No. 1422*, Transportation Research Board, pp.55-60.

18. Yeh, S.T., Gilmore, J.B. (1992), "Application of EPS for Slide Correction," *Proceedings of the Conference on Stability and Performance of Slope and Embankments - II*," ASCE, Berkeley, pp.1444-1456.

19. Allen, T.M., Kilian, A.P. (1993), "Use of Wood Fiber and Geotextile Reinforcement to Build Embankment Across Soft Ground," *Transportation Research Record No. 1422*, Transportation Research Board, pp.46-54.

20. Upton, R.J., Machan, G. (1993), "Use of Shredded Tires for Lightweight Fill," *Transportation Research Record No. 1422*, Transportation Research Board, pp.36-45.

APPENDIX A

TYPICAL SPECIFICATIONS

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

The following specifications are included in this Appendix.

- Expanded Shale - Caltrans
- Geofoam Fill Material - Colorado DOT (Revised)
- Foamed Concrete - New York DOT
- Wood Fiber - Washington DOT
- Shredded Tire - Oregon DOT
- Fly Ash - Delaware DOT

Expanded Shale - Caltrans Specification.

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

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

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

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

The lightweight fill may be constructed by dumping from trucks or by any other method approved by the Engineer. Lightweight fill shall be placed in lifts of a maximum of one foot in depth of uncompacted material. Compaction shall be obtained by a minimum of one complete coverage of a D 6 dozer or other tracked equipment having similar contact pressures.

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

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

Geofoam Fill Material - Utah DOT (Revised)

Description

The work provided herein consists of furnishing all material, and equipment and placing Geofoam Fill, referred to in this specification as expanded polystyrene (EPS), complete, as specified herein, shown on the plans, or as directed by the Engineer.

Reference Publications

Some or all of the publications referred to hereinafter form a part of this specification to the extent referenced. The publications are referred to in the text by the basic designation only. The latest edition of the referenced publication shall govern.

American Society of Testing Materials (ASTM) Publications:

ASTM Designation	Test Method for
C 203	Breaking Load and Flexural Properties of Block-Type Thermal Insulation
C 578	Rigid Cellular Plystyrene EPS Thermal Insulation
C 272	Water Absorption of Core Materials for Structural Sandwich Construction
D 1621	Compressive Properties of Rigid Cellular Plastics
D 1622	Apparent Density of Rigid Cellular Plastics

EPS shall be fabricated using feedstock manufactured into blocks having no more than five percent regrind content. Blocks shall have a height of at least 610 mm (24 inches), a width of 1.22 m (48 inches), and length of 2.44 (96 inches). All blocks shall be shop-trimmed as necessary so that all surfaces are smooth and flat, and are within tolerances of 0.5% of respective height, width and length dimensions. Additional field and/or shop-trimming and cutting will be required as necessitated by the geometry of the fill being constructed.

EPS blocks shall conform to the specified type category in ASTM C-578 and have the following physical properties:

Physical Property	ASTM Test Procedures	Accepted Value		
		Type VIII	Type II	
Density	D 1622	196 (1.25)	236 (1.50)	N/m^3 (pcf)
Compressive Resistance	D 1621	90 (13)	103 (15)	kN/m^3 (psi) Minimum @ yield or 10% Deformation
Flexural Strength	C 203	207 (30	276 (40)	kN/m^3 (psi) Minimum
Water Absorption	C 272	3	3	% Less than by volume

(Note: The engineer should refer to table 10 for project specific properties of each type specified. The above types were used on the Utah I-15 construction)

The EPS shall contain a flame retardant additive and shall have UL Certification of Classification as to External Fire Exposure and Surface Burning Characteristics. EPS should be considered combustible and should be exposed to open flame or any source of ignition. EPS shall be treated to prevent insect attack and shall be protected from vector intrusion. The manufacturer shall present proposed treatment methods to the Engineer for review and approval.

The Contractor shall furnish the Engineer a certified test report from the producer showing all data required to indicate compliance with the specifications.

Sampling and Testing

Quality assurance testing and sampling to monitor the conformance of the EPS fill with the specification requirements, will be completed as directed by the Engineer. Density and geometry (dimensional tolerances) testing shall be conducted using full-size blocks. Blocks in conformance with contract requirements can be used to make required fills.

Testing to monitor the quality of the EPS shall be done at the discretion of the Engineer. Engineer has the right to random sample the manufacturing plant. If any block does not conform to the physical requirements or if it is damaged in any way, it may be rejected by the Engineer.

Protection

The Contractor shall prevent damage to the EPS during delivery, storage, and construction. Prior to delivery of EPS fill to the project site, the Contractor shall review and be thoroughly knowledgeable with the manufacturer's care and handling recommendations. Any EPS fill to be exposed to such light for more than 90 days shall be covered with opaque material which will prevent ultraviolet light degradation. Any damage to the EPS resulting from the contractor's vehicles, equipment, or operations, shall be replaced by the Contractor.

Placement of embankment will require special procedures and careful selection of appropriate construction equipment to prevent damage to the EPS fill. No heavy construction equipment or vehicles shall be allowed directly on the EPS. EPS must be protected from petroleum based solvents such as gasoline and diesel fuel.

Embankment over the side slopes of the EPS fills shall be placed starting at the bottom of the slope in such a manner as to prevent damage to the EPS. Finished EPS on side slopes shall have a minimum of 0.6 m embankment cover.

The embankment material, in areas beyond the lateral distances necessary to protect EPS from damage, shall be compacted per the I-15 Corridor Specifications Section 225. The intent of this requirement is to minimize the uncompacted zone around the geofoam wall.

Subgrade Preparation

1. Clear and grub site.

2. Dewater site as required.

3. Place a leveling course of PV drain blanket material (See Section 227 of the I-15 Corridor Specifications for gradation) over the prepared surface, 200 mm (8 inch) thickness minimum. Level ± 10 mm over 3 meters (1/2" per 10') horizontal.

Placement

EPS fill shall be placed to the lines and grades shown in the plans and as directed by the Engineer. The surface of a layer of EPS blocks to receive additional EPS blocks shall be constructed with a variation in surface tolerance of no more than 20 mm in any 3 m interval. All blocks shall accurately fit relative to adjacent blocks. No gaps greater than 20 mm will be allowed on vertical joints. The finished surface of the EPS fill beneath pavement sections shall be constructed to within the tolerance of zero to minus 60 mm of the indicated grade. The finished surface of the EPS fill on side slopes that receive soil cover shall be constructed to within a tolerance of plus 90 mm to minus 90 mm of the indicated grade.

Blocks placed in a row in a particular layer shall be offset 0.6 m (2.0') relative to blocks placed in adjacent rows of the same layer as shown on the plans. In order to avoid continuous joints, each subsequent layer of blocks shall be rotated on the horizontal plane 90 degrees from the direction of placement of the previous layer placed. Connector plates should be placed between horizontal layers of blocks. Blocks shall be cut using a saw or hot wire.

Because of the light unit weight of the EPS fill, it is the Contractor's responsibility to provide temporary weighing and/or guying as necessary until all the blocks are built into a homogeneous mass, and the pavement section as well as any soil cover are in place.

Connectors

Connectors shall be galvanized steel multi-barbed connectors. Each connector shall have a lateral holding strength of at least 60 lbs when tested with ASTM C578 Type I EPS, with a safety factor of two.

Install a minimum of 3 connectors for each 1.2m x 2.4m Section of geofoam material or as shown on plans or directed by the Engineer. Press firmly into the rigid foam until the connector is flush with the surface. Position the next foam block as specified and seat firmly before placing subsequent blocks.

Payment

Expanded polystyrene fill will be measured by the cubic meter in its final position in the roadway. The pay unit shall be cubic meter in place.

1. Description. This work shall consist of furnishing and placing lightweight concrete fill of the appropriate type at the locations indicated on the plans and where directed by the Engineer. The work shall be done in accordance with these specifications and in conformity with the lines, grades, thicknesses and typical sections shown on the plans or established by the Engineer in writing.

2. Materials

A. Materials shall meet the requirements of the following:

Materials	Subsection
Portland cement (Types I, II or III)	701-01
Water	712-01
Admixtures	711-08
Foaming agent	(See Below)

The foaming agent shall conform to the requirements of ASTM C-869. Foaming Agents that are on the Approved List issued by the Department's Materials Bureau shall be accepted at the site on the basis of the brand name labelled on the Foaming Agent container and certified documentation issued by the supplier.

A foaming agent not on the Approved List will be evaluated based on submitted information and sample testing by the NYSDOT Materials Bureau (minimum of six months). For each class of material submitted for evaluation, specimens will be required for testing of compressive strength, air-dry density, freeze-thaw and water absorption characteristics and other testing as deemed appropriate. For detailed information contact the NYSDOT Materials Bureau.

B. The lightweight concrete fill shall conform to one of the following types as specified on the plans and shall be mixed in accordance with the recommendations of a representative of the supplier of the foaming agent:

Type	Maximum Cast Wet Density kg/m^3	Minimum Compressive Strength (28 days) kPa
A	480	276
B	670	690

The Contractor shall be responsible for designing the mix so that each type of lightweight concrete fill meets the corresponding criteria listed above.

C. During the initial placement of the lightweight concrete fill, the density will be determined at the point of replacement and the mix shall be adjusted by the Contractor, as required, to obtain the specified cast wet density. Thereafter, the density will be monitored by the Engineer at 30 minute intervals during placing and the Contractor shall adjust his operations as necessary to maintain the specified Cast Wet Density.

Specimens for determination of the compressive strength will be taken by the Engineer at the point of placement. Sampling will be in accordance with Department procedures as follows:

1. Four representative samples (157mm x 305mm cylinders) shall be taken at the point of placement for each day's pour or each 100 cubic meters of material placed, whichever is more frequent. Samples shall be marked for clear identification and all pertinent field information will be recorded on the corresponding field report, Form BR 300, including the station limits and elevation limits of the placement. Slump and air content shall not be measured.

2. Samples shall be obtained by overfilling the cylinders by pouring concrete down the insides of the cylinders, allowing air to escape during filling. DO NOT ROD THE SAMPLES. The sides and bottom of the cylinder molds shall be tapped to close any accidentally entrained air voids. Strike off the top of the cylinder (not more than three times) and cover.

3. Samples shall be placed in a location where they will not be disturbed nor subjected to temperatures below 7°C or above 29°C. Excessive handling may damage these test cylinders.

4. After 24 hours the Engineer will ship the cylinders along with the corresponding field test reports to the Materials Bureau for storage and testing. At 28 days the cylinders will be compression tested.

Failure to meet the cast wet density or the strength criterion for the appropriate type may require removal and replacement of that entire lift, and all overlying lifts, at the Contractor's expense based on an engineering evaluation by the Soil Mechanics Bureau.

3. Construction Details. Mixing and placing operations shall be under the supervision of the Engineer. A representative of the supplier of the foaming agent shall be on site during the initial placement and at such times as requested by the Engineer to advise the Contractor on his operation. The lightweight concrete fill shall be placed in lifts not to exceed 0.6 m unless otherwise approved by the Engineer. Subsequent lifts shall be placed only after a minimum 12 hour waiting period has been observed.

   The lightweight concrete fill shall be placed on supporting surfaces which have been cleaned of loose debris, sand, dust, or other foreign materials to the satisfaction of the Engineer. Surfaces against which the lightweight concrete fill is to be placed shall be free of ice, snow or standing water and shall be at a temperature of 1°C or higher.

   If the ambient air temperature is at or below 7°C or is expected to fall below this temperature during the curing period of the lightweight concrete fill, the Engineer may require that the exposed surfaces be covered with insulating blankets or hay, bat insulation, or solid or sprayed foam. The insulating material shall meet the requirements of Subsection 711-07, Form Insulating Materials for Winter Concreting.

4. Method of Measurement. The quantity of lightweight concrete fill in cubic meters shall be computed from payment limits shown on the plans or from revised payment limits established in writing by the Engineer at the time of construction.

5. Basis of Payment. The unit price bid for this item shall include the cost of furnishing all equipment, labor, and materials necessary to complete the required work. All costs for insulating, including the insulating material, shall be included in the price bid for lightweight concrete fill.

Wood Fiber - Washington DOT

Wood Fiber in Place

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

Materials

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

Construction

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

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

Measurement

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

Payment

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

Shredded Tire Oregon DOT Specification

The lightweight fill shall consist of chipped rubber tire pieces meeting the following specifications:

A. 80 percent of the material (by weight) must pass a 200-mm screen.

B. At least 50 percent of the material (by weight) must be retained on a 100 mm screen.

C. All pieces shall have at least one sidewall severed from the face of the tire.

D. The largest allowable piece is ¼ circle in shape or 0.6 m in length, whichever is the lesser dimension.

E. All metal fragments shall be firmly attached and 98 percent embedded in the tire sections from which they were cut. No metal particles shall be placed in the fill without being contained within a rubber segment. Ends of metal belts and beads are expected to be exposed only in the cut faces of some tire chips.

F. The lightweight fill material supplied shall weigh less than 3.5 kN/m^3 truck measure.

G. Unsuitable material delivered to the project shall be rejected in truckload quantities and removed from the site at no cost to ODOT.

H. ODOT, by use of this material, does not absolve the supplier of the responsibility of proper disposal of the lightweight rubber fill material if the section should fail to perform as expected.

NOTE: These specifications do not adequately describe the material that has been experimented with to date. Many of the shredded segments are incompletely severed from each other which results in long supple rubber pieces. Those pieces could be pulled through a 200 mm screen but would not necessarily fall through one. The processing for the chips has been a single pass through a rotary slow speed shear shredder equipped with 50 mm cutters. As long as the cutter tolerance is kept close and new cutters are installed as they wear, this produces an acceptable product for road construction. A hammermill process is not acceptable.

Fly Ash Delaware DOT Specification

Description

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

Materials

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

Drainage Foundation

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

Hauling

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

Construction Methods

The fly ash embankment shall be constructed upon a 1 m thick foundation consisting of 0.3 m of washed sand placed in a single layer and compacted with 2 passes of a vibrating roller, and a minimum of 0.6 m of compacted borrow Type A, meeting the requirements of Section 209. The fly ash shall constitute the embankment core and be contained within dikes formed of compacted Borrow Type F, Section 209, at the embankment slopes as shown on the plans. The fly ash shall be delivered to the project with a moisture content within 4 percent of optimum moisture but not exceeding optimum moisture by greater than 1 percent. The fly ash shall be spread out, tracked in with a dozer and compacted immediately after it has been deposited. Stockpiling of the fly ash in the embankment areas or in other areas outside of its source will not be permitted. The placement of fly ash on wet, unstable or frozen materials will not be permitted. The fly ash shall be placed in a level uniform cross section not exceeding 200 mm in depth loose measurement. It shall be deposited and spread parallel to the centerline and the layers shall extend the full width of the embankment. The loose lift shall be tracked in with a dozer prior to compaction. A water truck shall be available to add water prior to compaction if needed, in order to obtain the desired range of moisture.

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

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

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

The Contractor shall establish grade stakes in the field and submit for approval by the Engineer the final grades of the top of the drainage foundation layers (top of Borrow, Type A and Section 804 fine aggregate) and the top of the fly ash prior to placement of the subsequent layer of material. A minimum of ten working days shall be allowed for review by the Engineer. Adjustments to the final grades as directed by the Engineer shall be performed by the Contractor with all costs incidental to this item.

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

Compaction Equipment

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

Density and Moisture Control

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

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

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

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

Method of Measurement

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

Basis of Payment

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

VIBRO-COMPACTION

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CHAPTER 1

DESCRIPTION AND HISTORY

1.1 DESCRIPTION

Vibro-compaction is a ground improvement technique which uses specially designed probe-type, depth vibrators for in-situ densification of loose sands and gravels, as shown in figure 1. Originally called Vibroflotation, the process was accomplished with water jetting, hence the name.[1] With the advent of higher horsepower equipment with higher amperage tolerances, a dry operation became possible. The majority of vibro-compaction projects, however, are accomplished by the jetting water (wet) method, and this technical summary reflects that tendency.

The mechanism of densifying granular, cohesionless soils with vibrators can be briefly described as follows: mechanical vibrations and simultaneous application of water nullify the effective stresses between the soil grains which are rearranged, unconstrained and unstressed under the action of gravity to the densest possible state, thus providing permanent compaction. In the immediate vicinity of the vibrator, the soil is saturated and liquefies locally and temporarily, under the influence of the vibrations.[2]

There are numerous applications of vibro-compaction, including densification of granular hydraulic fills, coastal plain sediments, glacial deposits, alluvial soils, and miscellaneous granular fills and/or deposits. Vibro-compaction allows the use of spread footings with bearing capacities of 240 to 480 kPa. Also, liquefaction potential can be reduced by vibro-compacting loose, granular soil so that it is densified beyond the threshold relative density for liquefaction to be triggered. In earth retaining problems, the process can be performed prior to wall construction to decrease active earth pressure and increase passive resistance as the relative density is improved. Generally, vibro-compaction can be used to achieve the following results:

- Increased soil bearing capacity, permitting shallow foundation construction.
- Reduced foundation settlements.
- Increased resistance to liquefaction.
- Increased resistance to shear movements.

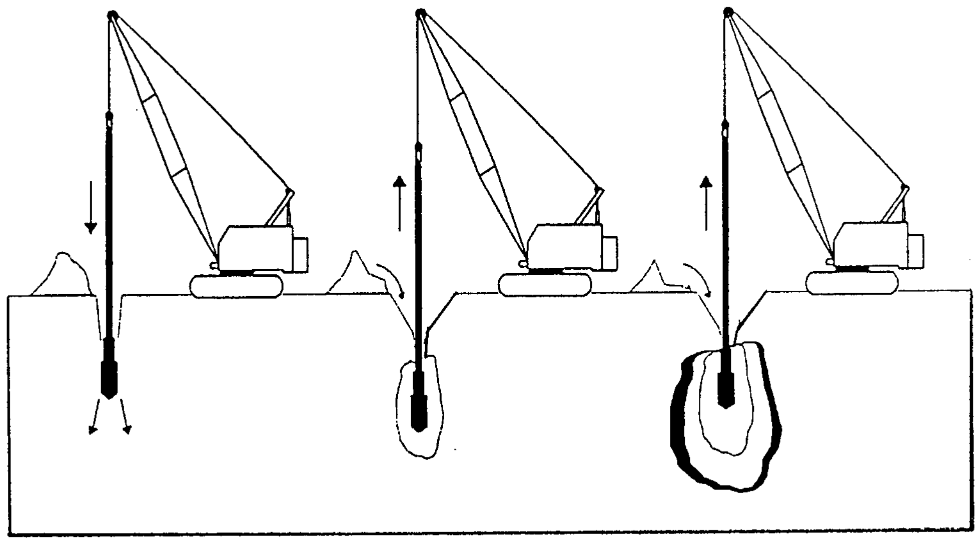


Figure 1. The vibro-compaction process.

- Reduced permeability.
- Increased density.
- Increased resistance to erosion and/or movement by water flow.
- Filling of voids in treated areas.

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

1.2 FOCUS AND SCOPE

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

1.3 HISTORICAL OVERVIEW

The fundamental concept of vibro-compaction ground treatment was developed in Germany during the early 1930s for compaction of variable and loose naturally occurring sand deposits to depths of 20 m. It was used in the densification of underwater sands for the seaport developments of northern Germany. Previous methods of compacting sand deposits consisted essentially of surface vibration or rolling. In order to overcome the limitations of these methods, a technique was developed whereby a metal tube or probe, which had within its lower end an electric motor driving an eccentric weight, was inserted into the ground. Vibrations were imparted into the ground as the tube was inserted to a required depth. To assist penetration of the vibrator, pressurized water was jetted through the tip of the probe. This original process was patented under the name of Vibroflotation.

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

There are several systems that have been historically identified as vibro-compaction. They include Vibroflotation, Terra Probe and the vibratory beam. Vibroflotation refers to compaction by means of a vibroflot which densifies with predominantly horizontal vibrations, while other techniques such as Terra-Probe, Vibro-Wing and Tri-Star or Y-Probe utilize a top pile vibrator which densifies with predominantly vertical vibrations, that normally requires closer spacing and is less efficient.[11,15]

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

1.4 SUGGESTED READING

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

Vibro-Compaction

- "Vibroflotation Compaction of Cohesionless Soils"[3]
- "Sand Compaction by Vibroflotation"[4]
- "The Development of Compaction Methods with Vibrators from 1976 to 1982"[5]
- "Vibratory Deep Compaction of Underwater Fill"[6]
- "Wando Terminal Ground Improvement Program"[7]

General Ground Improvement

- "In-situ Soil Improvement Technologies"[8]
- "In-Place Treatment of Foundation Soils"[9]
- "Soil Improvement: State-of-the-Art Report"[10]
- "Soil Improvement--A Ten Year Update"[11]
- "Ground Control and Improvement"[12]
- "In-Situ Deep Soil Improvement"[13]
- "Soil Improvement for Earthquake Hazard mitigation"[14]
- Ground Improvement, Ground Reinforcement, Ground Treatment: Developments 1987-1997"[15]

CHAPTER 2

DESIGN CONSIDERATIONS

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

2.1 APPLICATIONS

This technical summary focuses on the use of vibro-compaction as a solution to problems related to transportation projects. Well over 2,000 vibro-compaction projects have been completed in the United States, with about 10 percent being transportation related.

For transportation projects, vibro-compaction can be used to treat problems related to the following:

- Foundation soils beneath proposed buildings.
- Highway embankment fills.
- Tunnels - compaction of overburden soils.
- Densification of artificial tunnel islands.
- Mitigation of liquefaction potential for transportation applications:

    1) Compaction to stabilize pile foundations driven through loose granular materials.
    2) Densification for abutments, piers, and approach embankment foundations.

- Compaction of underwater embankment fills.
- Compaction in areas of potential cavities beneath embankments to presettle and fill such voids prior to construction of a structure.

2.2 ADVANTAGES AND DISADVANTAGES/LIMITATIONS

Advantages

As an alternative to deep foundations, vibro-compaction is usually more economical and often results in significant time savings. Loads can be spread from the footing elevation, thus minimizing problems from lower, weak layers. The risk of seismically induced liquefaction can be considerably reduced by densifying the soils with vibro-compaction. Vibro-compaction can also be a cost-effective alternative to removal and replacement of poor load-bearing soils. The use of vibro-compaction allows the maximum improvement of granular soils to depths exceeding 30 m. The vibro-compaction system is effective both above and below the natural water level.

Disadvantages/Limitations

The major disadvantage of vibro-compaction is that it is effective only in granular, cohesionless soils. When granular soils contain more than 15 percent silt and 2 percent clay, the resulting cohesion prevents the realignment of the sand grains and, consequently, the proper densification of the soil. The maximum depth of 30 m may be considered a disadvantage, but there are very few construction projects that will require densification to a greater depth.

Like all ground improvement techniques, a thorough soil investigation program is required. A more detailed soils analysis may be required for vibro-compaction than for a deep foundation project. This is because the vibro-compaction process utilizes the native soil to the full depth of treatment to achieve the end result. A comprehensive understanding of the total soil profile is therefore necessary. A vibro-compaction investigation will require continuous standard penetration tests, as well as some gradation tests to verify that the soils are suitable for vibro-compaction.

2.3 FEASIBILITY EVALUATIONS

a. In-situ Soil Gradations

The suitability of a soil for vibro-compaction methods depends mainly on its grain size distribution, as shown in figure 2. Soils with grain size distribution curves lying entirely on the coarse side of the hatched zone are readily compacted with depth vibrators. If the grain-size distribution curve falls in the hatched zone, it is advisable to backfill with stone in lieu of sand during the compaction process to improve the contact between the vibrator and the treated soil. The soils with grain size distribution curves partly or entirely on the fine side of the hatched zone are not readily compactable by vibro-compaction. However, these soils can be improved by vibro-replacement, as described in the companion technical summary "Ground Improvement by Stone Columns."

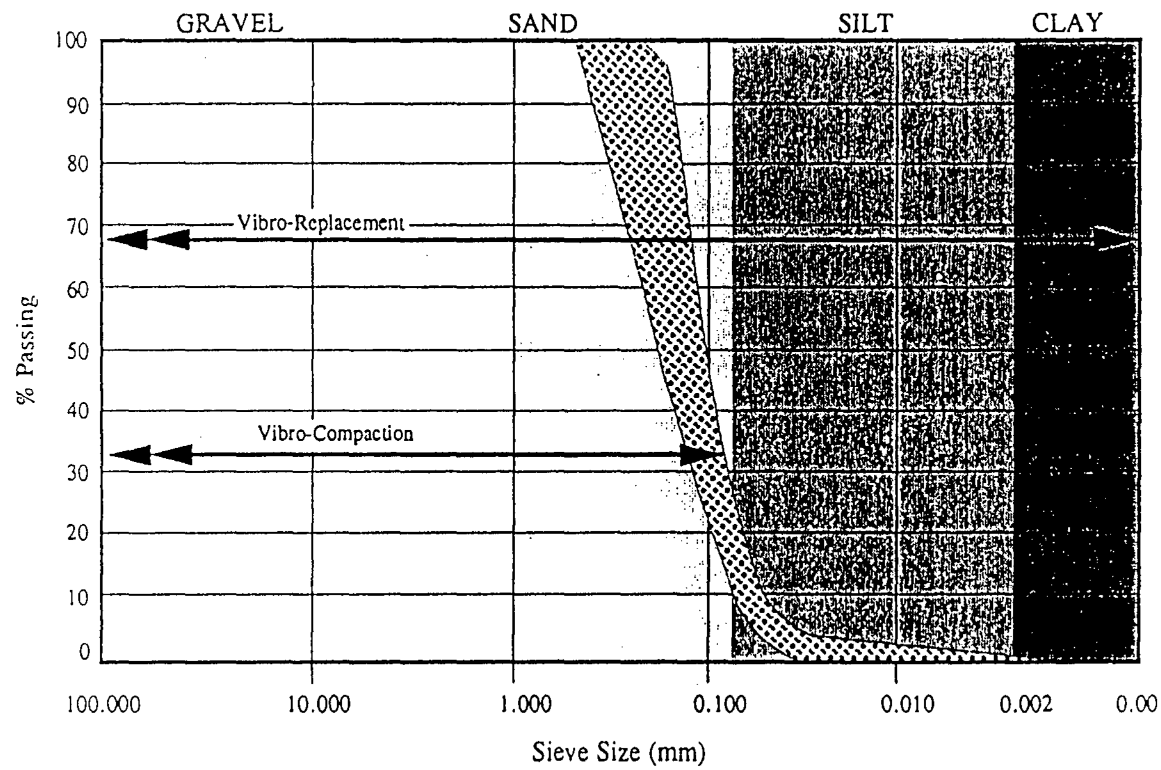


Figure 2. Range of soil types treated by vibro-compaction and vibro-replacement.

For non-cohesive soils with natural dry densities less than their maximum dry densities, the influence of vibrations will result in a rearrangement of their grain structures. Under the influence of induced vibrations, the inter-granular forces between the grains in non-cohesive soils are temporarily nullified. The grains are then rearranged, unconstrained and unstressed under the action of gravity to a more dense state. The void ratio and compressibility of the soil treated by vibratory means will be decreased and the angle of shearing resistance increased. The treated, compacted soil is capable of sustaining higher bearing pressures for the same settlements as the untreated soil, and undergoes smaller settlement for the same bearing pressure, with the settlement generally being only elastic. The achievable reduction in void ratio depends on grain shape, soil composition and vibration intensity. (2)

By advancing the vibrator to the desired level and withdrawing it from the ground in a specific manner, the granular soils are compacted by the horizontal vibration forces. A compact soil cylinder is thus formed, with the diameter determined by the grain size distribution, the soil density, and the vibrator characteristics. By arranging compaction points in suitable patterns, soil masses can be compacted homogeneously.

The increased density of the granular soils results in the downward movement of the soil around the vibrator and creates a cone-shaped depression at the surface. This depression must be continuously infilled with granular fill material. If on-site material is used, then the original ground surface will be lowered. Alternatively, ground level can be maintained by adding imported granular fill material which is compacted simultaneously with the natural soil.

The vibro-compaction process subjects the soil mass to high accelerations during compaction. These levels of dynamic strain are unlikely to be repeated, even under earthquake loading. Provided that the design earthquake criteria are not exceeded during a seismic event, the treated ground can be expected to perform as designed.

Soil compaction, as achieved in the vibro-compaction process through the rearrangement of soil particles, is not possible in cohesive, fine-grained soils. The cohesion between the particles prevents rearrangement and compaction from occurring. Stabilization of cohesive and very fine-grained and cohesionless silty soils can be achieved by vibro-replacement. (12) Figure 2 illustrates the range of soils that can be treated by vibro-compaction and vibro-replacement.

b. Environmental Considerations

The dry method of vibro-compaction is only viable in clean, sandy, fully saturated soils . The great majority of vibro-compaction projects are therefore accomplished by the wet method. Although the vibro-compaction technique is used for densifying primarily granular soils, the jetting water effluent will nevertheless require temporary construction provisions to contain and dispose of any silt and clay in the effluent. With current awareness of potential environmental problems, geotechnical exploration programs should include not only the classification of the soil type and location of groundwater, but also the examination and classification of any potential contaminants in the soil and groundwater. If contaminants are uncovered in the original exploration program, a determination should be made as to whether they can be treated at the site during the vibro-compaction program. If this is not the case, then an alternative densification program such as the dry bottom feed stone column technique (which does not produce jetting water effluent) or another solution, should be considered.

2.4 ALTERNATIVE DEEP COMPACTION METHODS

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

Compaction or Displacement Piles

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

Frankipaction

The Frankipaction system is essentially an adaptation of the Frankipile to achieve compaction. The Frankipile is a driven, enlarged-base pile. The pile casing is positioned on the ground and a charge of dry concrete is placed within the bottom of the casing. A drop hammer is used to drive the concrete, which then forms a dense plug that penetrates the ground, dragging the casing down with it. When the pile has reached the desired depth, the concrete end-plug within the casing is driven down and outward to form the pile base. Successive charges of concrete are then placed and compacted as the casing is withdrawn, forming the pile shaft.[16] In Frankipaction, sand or gravel is used instead of concrete, and a compaction pile is created. The major advantage of this system is the ability to put large volumes of stone in layers of soft or compressible soils to minimize settlements or potential shear failures of these layers while still achieving densification in the granular layers. Generally, for densification, vibro-compaction methods are more economical than Frankipaction.

Blast Densification

Blast densification to achieve densification has been used since the mid 1950s. While it is potentially economical compared with other systems, it is generally only useful where densification is necessary over large areas and at great depths (>30 m). As with dynamic compaction (described below) the technique has the inherent potential to have a negative effect on both the natural and man-made environment. Blast densification is more vulnerable to litigation than most other densification techniques and therefore requires a high degree of expertise and experience in its execution. Despite these drawbacks, the technique should be considered useful under the right circumstances. Washington State DOT has reported a blast densification project in which potentially liquefiable, loose debris avalanche deposit from the 1980 eruption of Mt. St. Helens, was densified by blasting techniques.[17]

Dynamic Compaction

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

Compaction Grouting

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

Vibratory Hammer Probe

The vibratory hammer probe method differs from the vibro-compaction method in that the vibrations are transmitted vertically down the attached pipe of a typical diameter of 0.8 m. The vibro-compaction method transmits a horizontal vibration over a distance of up to 4 m. The frequency of vibration of the vibratory hammer probe and the location of the vibrator result in a less effective densification process and therefore must be used on a significantly closer pattern spacing. Usually this results in higher overall cost for densification.

CHAPTER 3

CONSTRUCTION AND MATERIALS

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

3.1 EQUIPMENT

The equipment used to achieve the necessary densification are high-powered, probe-type vibrators ranging from 0.3 m to 0.45 m in diameter and 3 m to 5 m in length, as shown in figure 3. A set of rotating eccentric weights housed inside the probe is mounted on a vertical shaft. Vibrations (induced by rotating these weights) are produced close to the bottom of the unit. The rotating shaft is driven by a motor located within the casing, as shown in figure 4. The entire vibrator has a mass of about 1,800 kg. To drive this assembly, an electrically driven motor is usually employed, although hydraulic units are also used. The early units were driven by motors in the 20 to 60 kW range. However, relatively recent developments have introduced units producing up to 120 kW. The vibrations produced by these units are generated at the nose of the unit and, as a result of the rotation of the weights, emanate radially in the horizontal plane away from the unit. The larger units now in general use in the United States generate centrifugal forces in excess of 175 kN at frequencies ranging from 1,500 to 3,000 rpm. However, for vibro-compaction, vibrators operating at lower frequencies will *usually* produce better densification results than those operating at higher frequencies. This is because low frequency vibrators usually have a higher amplitude, which translates into a greater compactive effort. Additionally, the natural frequency of most densifiable soils is closer to 1500 rpm than to 3000 rpm.

Table 1 describes one contractor's first three generations of electrically driven vibrators, showing the continuing upgrade of the motor output, the amount of centrifugal force, and the increase in amplitude.

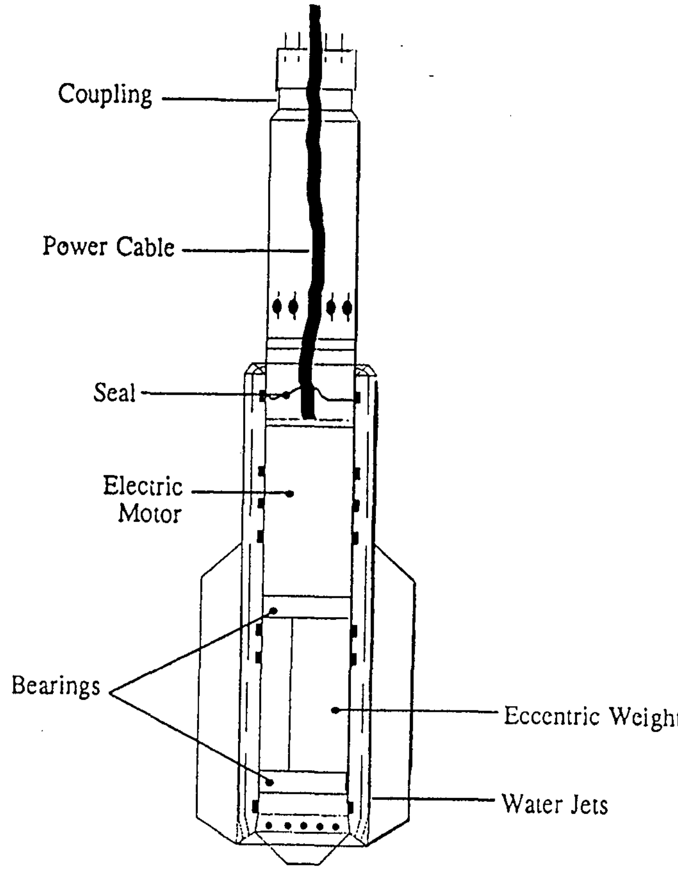


Figure 3. High-powered, probe-type vibrator utilized in vibro-compaction.

Figure 4. Cross-section of a typical vibrator.

Table 1. Specifications of several vibrators.

Type of Vibrator	**Motor Output (kW)**	**Speed (rpm)**	**Centrifugal Force (kN)**	**Amplitude 2-sided (mm)**
Torpedo Vibrator (1st Generation)	35-60	3,000	160	4
Mono-Vibrator (2nd Generation)	50	3,000	150	7
A-Vibrator (2nd Generation)	50-80	2,000 to 3,000	160	14
S-Vibrator (3rd Generation)	120	1800	206	17

Follower tubes of a similar or lesser diameter are attached to the vibrating unit in order to extend its length to allow treatment of soils at depth. The follower tubes are attached to the vibratory unit by means of an isolation coupling, thus preventing the vibrations from travelling up the follower tubes, negating the problem of energy losses at depth.

The complete assembly is supported from either a standard crane (figure 5), a specially built hydraulic crawler crane, or a crane that is mounted on a barge (figure 6), depending upon the site conditions.

The vibro-compaction operation necessitates the use of water or air jetting to facilitate the penetration of the vibrator and to densify the soil. Therefore, water or air feed hoses, as well as water or air pumps, are also required.

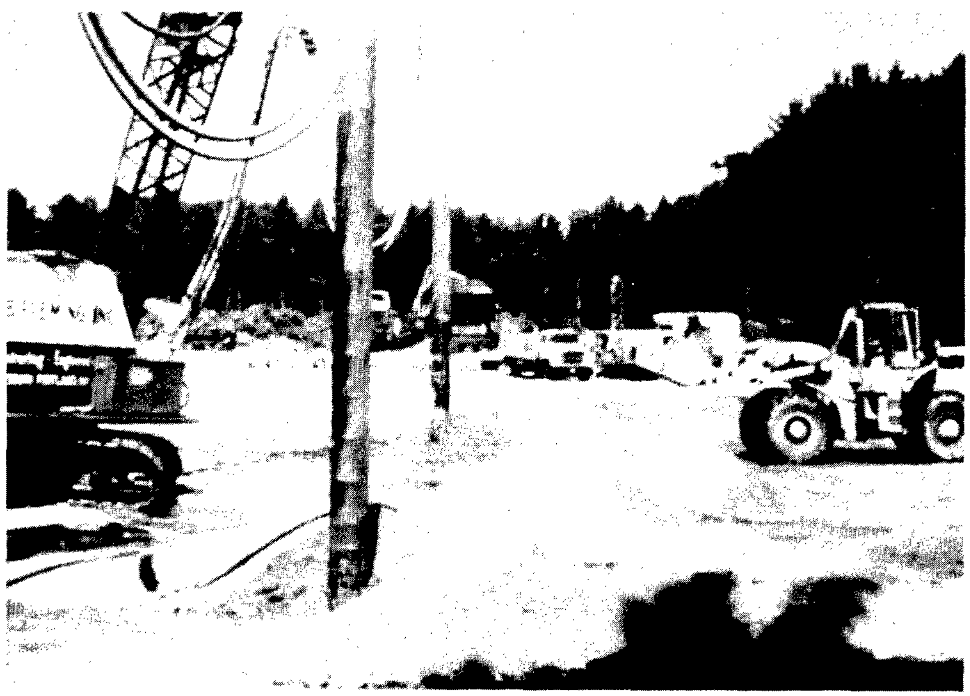


Figure 5. Vibrator suspended from a conventional crane.

Figure 6. Vibrator suspended from a barge-mounted crane.

3.2 MATERIALS

To achieve adequate compaction, it is necessary to supply sufficient backfill material to fill the void created by the densification process in order to transmit vibrations from the vibrator into the in-situ soil. Fine sands, coarse sands, rounded gravel, crushed stone, recycled aggregate and slag have all been used as backfill material. Slag has the advantage of being inexpensive in some locations, but does not settle as fast as other material with comparable gradation. Coarse materials with little or no fines make the best backfill. However, if the particle size becomes too large, the gravel will arch in the annular space between the follower tube and the void, preventing backfill from reaching the vibrating tip.

The suitability of the backfill appears to be a function of the backfill quantity that can accumulate around the vibrating tip in a fixed period of time. The backfill gradation is the most significant factor controlling the rate at which the backfill settles through the wash water and accumulates around the tip. A rating system has been developed to judge the suitability of backfill material for vibro-compaction based on the settling rate of the backfill in water and project experience.[3] This rating is dependent on a "suitability number" and is a function of the grain size diameters of the backfill material. The equation used in this calculation is as follows:

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

where D_{50}, D_{20}, and D_{10} are the grain size diameters, in millimeters, at 50 percent, 20 percent, and 10 percent passing the 74μ (#200 sieve). Table 2 explains the qualitative categories of backfill utilizing this rating system more precisely. The quality of backfill material affects the allowable withdrawal rate of the vibrator. Within reasonable limits, the lower the suitability number the faster the backfill will settle and the vibrator can be withdrawn and still achieve acceptable compaction. Backfill normally consist of material graded as sand or sand and gravel with less than 10 percent by weight passing the 74μ (#200) sieve, and containing no clay.

Table 2. Backfill evaluation criteria.

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

CHAPTER 4

DESIGN CONCEPTS

Design of a vibro-compaction program begins with a definition of the problem and the development of performance requirement for the improved soil. Depending on the type of project being designed, the prime consideration could be total or differential settlement, bearing capacity, or a seismic resistance requirement.

This chapter discusses problem definition and performance requirements for vibro-compaction, together with basic, empirically derived design layout patterns.

4.1 PROBLEM DEFINITION

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

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

If loose granular soil is identified as the problem, then densification by vibro-compaction will be a potential technical solution. Chapter 7 contains information to determine the economics of vibro-compaction and chapter 5 should be helpful in specification preparation. Table 3 indicates the relationship between penetration resistance from subsurface investigations and sand soil properties necessary in assessing the density of sand deposits.

Table 3. Penetration resistance and sand properties.

	Very Loose	Loose	Medium Dense	Dense	Very Dense
SPT N-value (blows/0.3m)*	<4	4-10	10-30	30-35	>50
CPT cone resistance (kg/cm^2)*	<50	50-100	100-150	150-200	>200
Equivalent Relative Density (%)**	<15	15-35	35-65	65-85	85-100
Dry Unit Weight (kN/m^3)	<14	14-16	16-18	18-20	>20
Friction Angle (*)	<30	30-32	32-35	35-38	>38
Cyclic Stress Ratio Causing Liquefaction	<0.04	0.04-0.10	0.04-0.10	>0.35	

* At an effective vertical overburden pressure of 100 kPa.

** Freshly deposited, normally consolidated sand.

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

- The gradation of the in-situ soils, including silt and clay content.
- The existing relative density, or looseness, of the in-situ soils.
- The required density improvement necessary to solve the project's requirements and, once determined, whether this improvement is feasible. If the specified densification is more than required by the project, the increased densification will increase the construction costs as compaction points will be required at closer spacings.

4.2 DESIGN CONCEPTS

The significant engineering properties of a granular soil - compressibility, shear resistance, permeability, resistance to dynamic loading are largely dependent on the state of compaction, typically expressed in terms of relative density. The term "relative density," or Dr, is defined as follows:

$$Dr=\frac{W_n-W_l}{W_d-W_l}x\frac{W_d}{W_n}x100\% \tag{2}$$

where: W_n is the dry density of the soil in its natural state,
W_l is the dry density of the soil in its loosest state,
W_d is the dry density of the soil in its densest state.

In this calculation, W_l and W_d should be determined in accordance with AASHTO or ASTM procedures.

High relative density leads to high safe bearing capacity with low settlement. In the case of seismic loading, resistance to liquefaction in granular soil is a function of relative density. In earth retaining problems, active pressure decreases and passive resistance increases as relative density increases.

With vibro-compaction, the angle of internal friction is increased on average between 5 and 10 degrees, resulting in much higher shear resistance. The modulus of deformation is increased to approximately 95 MPa and in some cases higher, and consequently settlements are greatly reduced.

In addition to soil gradation, the area influenced (tributary area) by each compaction point for a specified relative density depends on the compaction method used and the specific characteristics of the vibrator.

Figure 7a shows the relationships between relative density, soil type and treatment area. It is unlikely, even for heavy loading, that it will be necessary to achieve a relative density above 85 percent. A relative density of 55 percent may be considered as the lower limit.

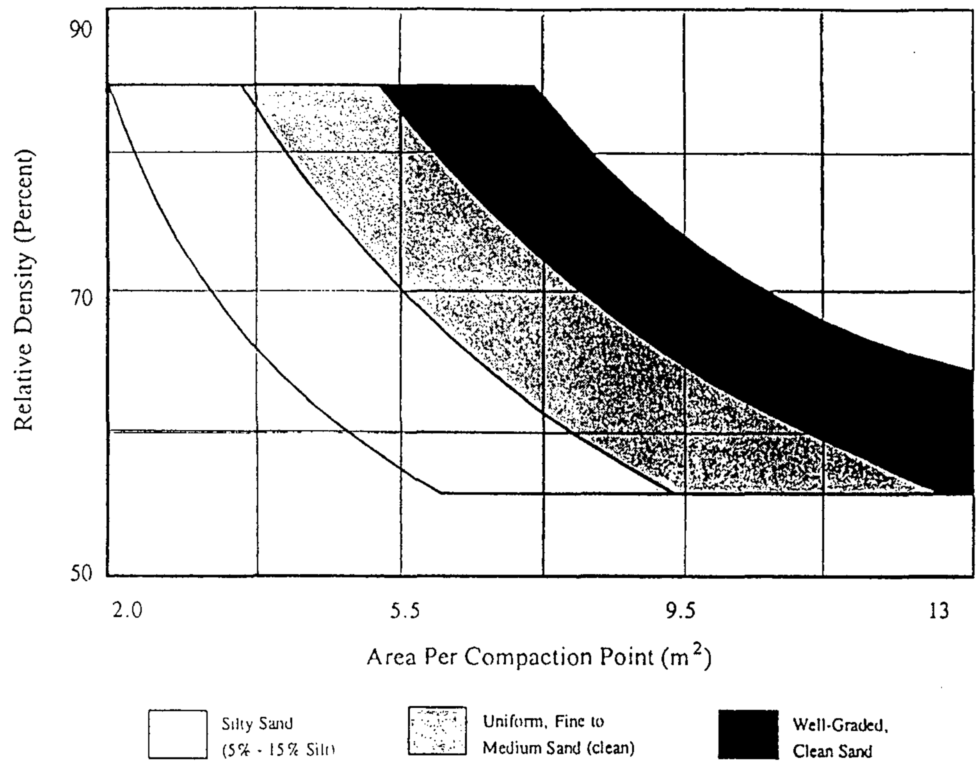


Figure 7a. Approximate variation of relative density with tributary area.

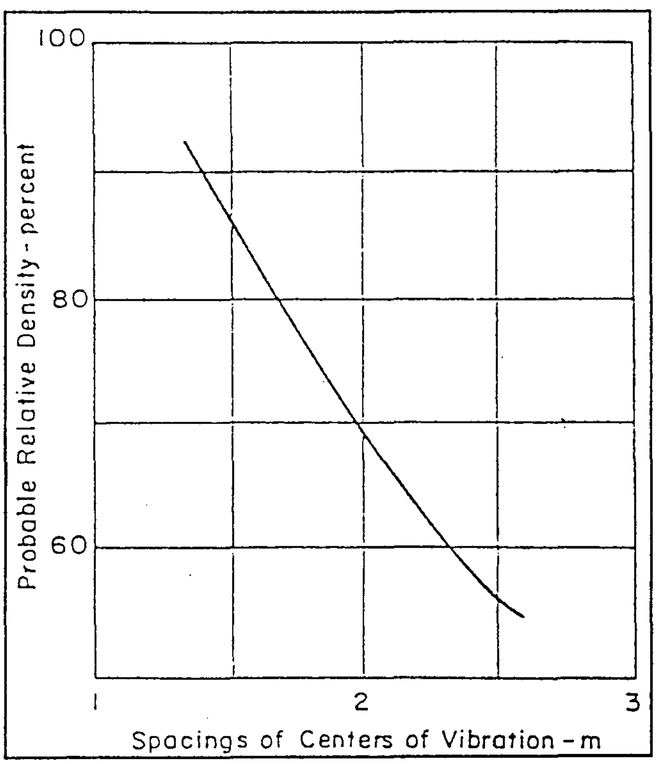


Figure 7b. Relative density vs. probe spacing

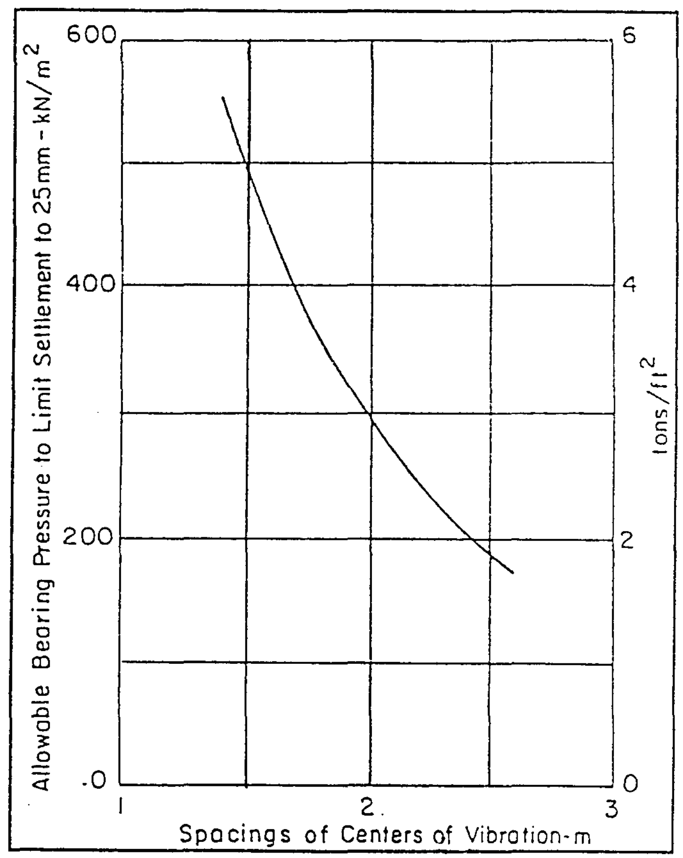


Figure 7c. Allowable bearing capacity vs. probe spacing

Figure 7b and 7c are preliminary charts useful in estimating the probable level of improvement that can be obtained by vibro-compaction. They are based on the lower bound soil gradation (silty sand) indicated in figure 7a.

The project designer is responsible for determining the requirements for the project with an appropriate safety factor and the best method of confirmation testing. For most vibro-compaction projects, the following performance criteria should be considered:

- 60 percent relative density for floor slabs, flat bottom tanks
- 70 percent relative density for column footings
- 80 percent relative density for machinery, and mat foundations

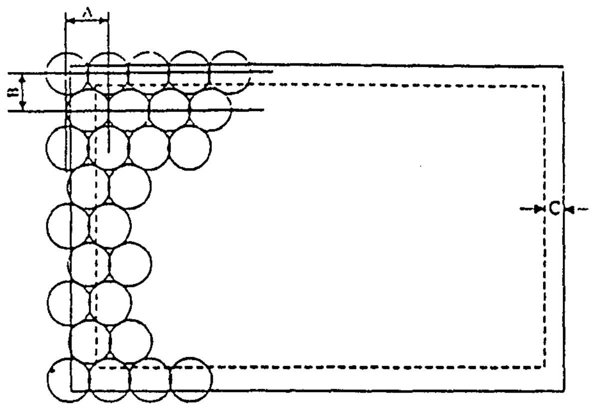
4.3 PROBE SPACING AND PATTERNS

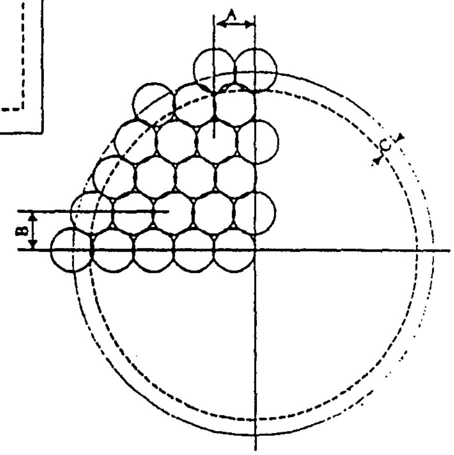
A typical vibro-compaction program is designed with various probe spacings and patterns. The distance between compaction points is critical, as the density change generally decreases as the distance from the probe increases. The influence of a single compaction point in clean sands extends from approximately 1.8 m diameter with a 22 kW unit, to approximately 2.7 m diameter with a 75 kW unit.

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

Given the in-situ soil gradation and relative density required to be achieved, the spacing of compaction points can be determined. Figure 8 shows typical area patterns and spacing for 80 percent relative density requirements.

Table 4 illustrates the number of vibro-compaction points typically required to achieve a relative density of 80 percent for the given column loads and footing sizes. Assuming an 80 percent relative density requirement, the vibro-compaction point spacing would typically be as shown in figure 9. The spacing of the vibro-compaction points will be wider for a lower relative density requirement.





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

* Based on performance requirement

Figure 8. Typical compaction point spacings for area layouts.

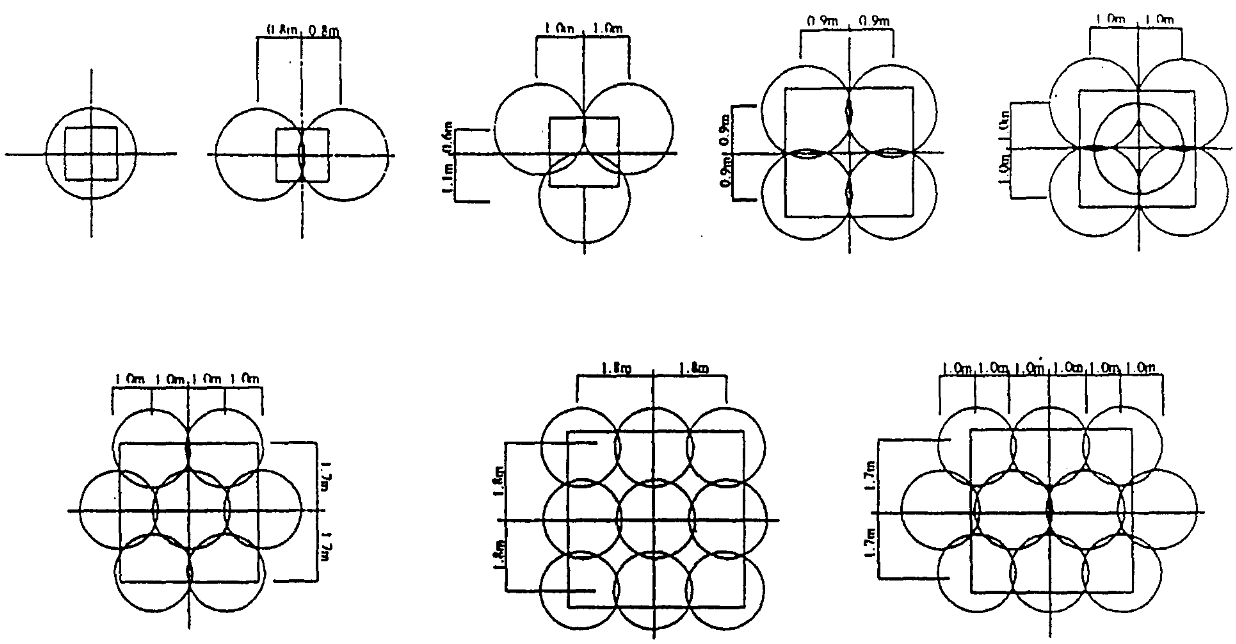
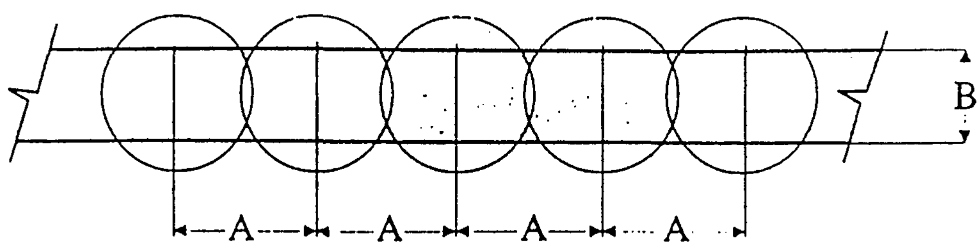


Figure 9. Typical compaction point layouts for column footings.

Table 4. Typical compaction point spacing parameters.

80 percent relative density		
Square Ftg Size (m)	**Col Load (kN)**	**No. of Compaction Points**
1.0	316	1
1.2	556	2
1.5	863	2
1.8	1237	3
2.1	1673	4
2.6	2447	5
2.9	3043	5
3.4	4043	7
3.8	5178	9
4.1	6010	9
4.6	7357	10

Vibro-compaction spacing for wall footings would typically be as shown in figure 10.



Relative Density**	A	Max. B*
80%	2.0m	0.7m

* Wider ftgs. require 2 or more rows.

** Based on performance requirement

Figure 10. Typical compaction point layouts for wall footings.

4.4 PERFORMANCE REQUIREMENTS

Performance requirements, such as total or differential settlement, bearing capacity or a seismic requirement, can all be related to a desired in-situ relative density. After compaction is completed, the effectiveness of compaction is normally evaluated to verify contractual compliance and to verify that the compacted soils will perform adequately under the intended loads. A number of testing methods are used, including Standard Penetration Testing, Cone Penetrometer soundings, or load tests. These methods, and their advantages and disadvantages, are described in chapter 5.

CHAPTER 5

CONTRACTING METHODS, CONSTRUCTION SPECIFICATIONS, AND CONSTRUCTION CONTROL

Vibro-compaction may be performed under either a method-type specification or a performance-type specification. Under a method-type specification, the specifying agency details a specific procedure and pattern spacing to achieve the required improvement. Bids are invited from contractors suitably equipped to perform the work. With this type of specification, the specifying agency assumes the risk, and a knowledge of the ground improvement technology is required. If this knowledge is not available within the specifying agency, outside expertise should be obtained.

Under a performance-type specification, the required end result is specified and the contractor assumes responsibility for achieving it. This approach does not require in-depth knowledge within the specifying agency. The contractor has the flexibility of selecting the procedure and pattern spacing to meet the design criteria.

Specifications and contracting procedures for vibro-compaction have changed significantly over the years. Where once the specifications stated a specific procedure and pattern spacing, variances in equipment and methods today favor placing the responsibility for achieving the required improvement on the contractor. Whereas the vibro-compaction method itself may still be specified, the contractor adopts the procedure and pattern spacing to achieve project objectives. Most vibro-compaction specifications today are performance based.

This chapter discusses contracting methods and quality control and inspection procedures. A guide to the preparation of a typical specification is included. Since the responsibility for achieving the design criteria for the ground improvement usually rests with the contractor under a performance specification, the focus of this chapter reflects this norm.

5.1 CONTRACTING METHODS

Most vibro-compaction projects require a certain degree of densification, which can be specified as follows:

- Minimum or average percent relative density.
- Minimum or average percent of maximum density.
- Minimum or average blow count.
- Minimum or average cone penetration resistance.
- Minimum size of gravel or sand column.
- Minimum amount of backfill material added.
- Minimum load bearing requirement.

All of the above have been used in past projects and for the most part have been successful. However, the best specification is one that allows for some variance of results within specified limits. Also, past experience has shown Standard Penetration Test blow counts to be misleading in certain stratifications and that specific percentage degrees of density are difficult to measure.

The technical literature has shown evidence that verification testing procedures can give misleadingly low results if performed immediately after densification, and that results can increase significantly with time.[19] Typically, a minimum of 48 hours is recommended before performing verification testing. The effectiveness of soil improvement with time after treatment should be considered in performing tests and interpreting test results.

It should be noted that qualified and experienced personnel using the proper equipment should be specified within vibro compaction specifications.

5.2 CONSTRUCTION SPECIFICATIONS

The following guide to the preparation of a performance-type specification can be used in the formulation of project-specific specifications. The format of this sample specification is deliberately generic: the responsible party should be inserted as appropriate when developing specifications for a particular project. Italicized terms or descriptions allow for flexibility to adapt to the specific requirements of the project to be improved by vibro-compaction. Where necessary, additional explanatory notes are included.

Guide To Specification Preparation For Vibro-Compaction

Scope of Work

This specification guide details the technical and quality assurance requirements for furnishing all supervision, labor, material, equipment, and related services necessary to perform all vibro-compaction soil improvement.

The work includes subsurface soil improvement by vibro-compaction and delivery and placement of all sand and/or stone backfill necessary in the improvement process. Soil improvement by the vibro-compaction method will be limited to the areas requiring *bearing/settlement/liquefaction potential/etc.* improvement as shown on the engineer's drawings for the following specified areas: *mat/column footings/wall footings/embankment slope/pavements, etc.*

The extent of the work should be shown on the engineer's drawing. *Drawing no., issue date, and title should be provided here.*

Qualifications

The contractor performing the vibro-compaction must have a minimum of 3-year experience record documenting 10 recent, successful projects completed. The Site Superintendent should have a minimum 3-year experience record of performing on-site supervision of vibro-compaction work. Documentation substantiating this experience should be provided to the engineer 30 days prior to beginning the vibro compaction work.

Requirements

- Backfill Material

It may be necessary to maintain site elevations during the vibro-compaction work. Restoration, if necessary, of site elevations by backfilling should be accomplished using procedures and materials as specified elsewhere.

Backfill should consist of material graded as sand or sand and gravel with less than 10 percent by weight passing the 74μ (# 200) sieve, and containing no clay.

- Equipment and Procedures

Specific equipment and construction procedures to achieve the specified criteria in a performance specification are left to the contractor. However, the following general guidelines are identified:

> The contractor should use a down-hole vibrator capable of providing at least 60 kW and 130 kN of force. After penetration to the treatment depth, the vibrator should be slowly retrieved in 0.3 to 0.5 m increments to allow backfill placement.
>
> The vibrator should re-penetrate each backfill increment into a recently treated depth interval to observe amperage build-up or increase. Amperage build-up and backfill quantities are contingent on the type of vibrator, type of backfill, in-situ soil conditions, and contractor's procedure. Discussion between the engineer and contractor should be conducted prior to beginning the vibro-compaction regarding individual equipment capabilities and expectations.

- Tolerances and Acceptance Criteria

The densification achieved and/or engineering properties of the subsoil mass after vibro-compaction is dependent on many variables. These include the densification technique employed, the depth of treatment, the type and quality of the equipment mobilized, the experience of the contractor, the backfill material utilized, the procedure used by the contractor, and the engineering and management support provided by the contractor.

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

SPT N Value	17-25 blows/0.3 m
CPT Resistance	7,660 to 19,150 kN/m^2
Relative Density	65 to 85 percent

Specific target criteria should be carefully chosen to provide an acceptable density. An averaging technique over a one- to one-and-a-half meter depth interval is often employed to account for anomalies in the soil mass.

It must be understood that vibro-compaction will not significantly improve soil containing over 15 percent silt and 2 percent clay.

In general, the vibro-compaction program should provide densification of the underlying soils as necessary to provide the following:

1) a relative density of ____ according to __________. [a correlation such as that developed by either Gibbs and Holtz, Schmertmann or Seed is typically specified.]

2) an ultimate bearing capacity of ___ .

3) total settlement limited to ___ .

4) differential settlement limited to _____ over a distance of _____.

5) an average corrected N' value of ___ for SPT testing.

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

The work should consist of densification of the underlying soils to a depth of __ *or 2.0 times the footing width* as measured from the footing base.

- Restrictions

Provisions should be made for obtaining all State and municipal permits and conforming to all State and local regulations.

[Environmental and Utility restrictions should be listed within this section]

Quality Assurance

- Testing and Inspection

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

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

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

- Documentation

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

A daily log should be submitted to the engineer by the contractor. This should include a recording of compaction point number, start/finish time of compaction point, depth of treatment, approximate backfill quantities per compaction point, and indication of relative ammeter increases for each point.

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

- Measurement and Payment

Performance specifications should delineate the area and depth of densification as well as the criteria to be achieved. On this basis, the experienced contractor can evaluate the subsurface and project conditions to provide a lump sum price for the work. [Under a performance specification, the lump sum price would typically include testing.] Unit area or volume prices should also be provided to address changes in plan quantities.

Mobilization/demobilization are typically presented as separate lump sum figures.

5.3 QUALITY CONTROL AND INSPECTION

The quality control plan and inspection activities are developed well in advance of the vibro-compaction work. The duties of the contractor and the owner/engineer with respect to QA/QC are dependent on the type of specification under which the work is being accomplished.

Under a method specification, development of the QA/QC plan, review of plans and specifications, and acceptance of backfill material are the responsibility of the owner's engineer. During performance of the work, the owner's engineer is responsible for on-site inspection and testing. *(Under a performance specification, these latter responsibilities lie with the contractor)*. The contractor provides the owner's engineer with testing results to verify that improvement criteria are being met. In the latter case the owner is obligated to perform independent testing. This should be based solely on the results of the contractor.

Under either type of specification, the inspection process during the actual compaction process should include the following:

(1) Verification that probe penetration depth is acceptable.
(2) Verification that probe withdrawal rate is acceptable.
(3) Monitoring the probe penetration rate to obtain a rough indication of the type and density of soil penetrated.
(4) Verification that compactions are at the proper locations,
(5) Monitoring the volume of backfill added to obtain an indication of the densities achieved.
(6) Verification that backfill gradation is acceptable.
(7) Monitoring of ammeter or hydraulic pressure readings to verify that the build-up is sufficient.
(8) Verification that the probes are operating at appropriate speeds.
(9) Verification that induced vibrations are not excessive when operating close to existing structures.

During the compaction process, the adequacy of compaction is periodically verified for quality control and acceptance purposes. This check is to verify contractual compliance and that the compacted soils will perform adequately under the intended loadings. A number of methods are used, including borings with SPTs, static CPTs, measurement of the surface subsidence, density measurements on undisturbed samples, and downhole nuclear densimeters. Each method has certain advantages and disadvantages.

The inexpensive SPT is the most widely available method and the most widely used. However, it is also the least reliable method for estimating potential settlements, bearing capacity, and relative density of the compacted soils. SPT resistance N values are variable depending upon a number of factors. There is also significant scatter in the correlations of SPT resistance with relative density and with the soil properties needed to estimate settlement and bearing capacity. In addition, if data are obtained before pore pressures have dissipated, the penetration resistance will not be representative of the actual degree of soil improvement. SPT data are usually taken at 1.5 m intervals, which is inadequate to properly evaluate the vertical variability of the vibro-compaction. However, this can be overcome by specifying continuous sampling, as is frequently the case.

The static CPT overcomes most of the disadvantages encountered with the SPT and is considered the best available QA/QC method. The cone resistance is particularly advantageous since it is relatively inexpensive and can be used directly to estimate settlements in compacted areas. The cone resistance, however, will underestimate the degree of improvement if excess pore pressures are present.

Measurement of surface subsidence is an excellent way of monitoring the average increase in relative density when the fill material is obtained from the compacted area. This method can also be used to check compaction of large areas if the quantity of imported fill is known. As a practical matter, it is difficult to accurately verify compaction achieved for footings with this method, and it is not possible to check for the minimum compaction achieved.

Downhole nuclear densimeters offer an alternate method for verifying final densities, but have not been used enough to establish their advantages and disadvantages relative to vibro-compaction. With this method, a small diameter aluminum pipe is placed in the ground to the planned compaction depth prior to compaction. Before and after compaction, a site calibrated nuclear probe is lowered down the casing to obtain a continuous density-moisture-content profile. This method indicates the density within approximately 150 mm of the aluminum pipe.

CHAPTER 6

CASE HISTORIES

The vibro-compaction technique is used to achieve a variety of design objectives. The case histories selected for this chapter represent different applications within the transportation industry.

6.1 DENSIFICATION OF GRANULAR SOILS

I-90 Mt. Baker Ridge, Seattle, WA[21]

Environmental considerations played a major role in an extensive improvement and expansion program for the I-90 corridor through the Mt. Baker Ridge area in Seattle, WA. With stretches of the improved interstate designed to carry 50,000 vehicles each way daily, the impact on residential communities was alleviated by deep-cut construction accommodating covered roadways. The roadway structures would support landscaped parks, effectively reclaiming these construction areas.

Massive pier footings on grade were required to support the covered roadway cross-section. In addition to providing vertical support for the cross-section, the footings were also designed to carry the lateral load of embankment soils placed behind the wall. For units 9 and 10 of the 790-m-long roadway section abutting the Mt. Baker Tunnel, the 91.5-m-long by 9.0-m-wide footing was to be placed directly on soils previously placed for the existing highway embankment. Originally, the footing design assumed a 285 kPa allowable bearing on to the fill soils. However, subsequent geotechnical investigation determined that the loose to medium-dense, silty, gravelly, fine-to-medium sand fill (approximately 6 m depth) could not support the 285 kPa loading without extensive settlement. Washington Department of Transportation engineers, in conjunction with their consultants, considered both deep foundations and in-situ soil improvement. Based on time and cost considerations, vibro-compaction was chosen as the best solution.

To meet densification criteria, stone backfill was specified for the vibro-compaction process. At 540 compaction points, a 120 kW vibrator densified the soils to a depth averaging between 4.5 and 6.0 m, as shown in figure 11. This treatment depth allowed for densification/reinforcement through the existing soils and through the loose upper zone of the underlying sandy glacial deposit. The area of treatment included a 3.0 m to 6.0 m perimeter around the entire footing. The compaction points were spaced on a 1.8 m grid pattern with the design intent to limit settlement to a specified requirement of 19.1 mm.

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

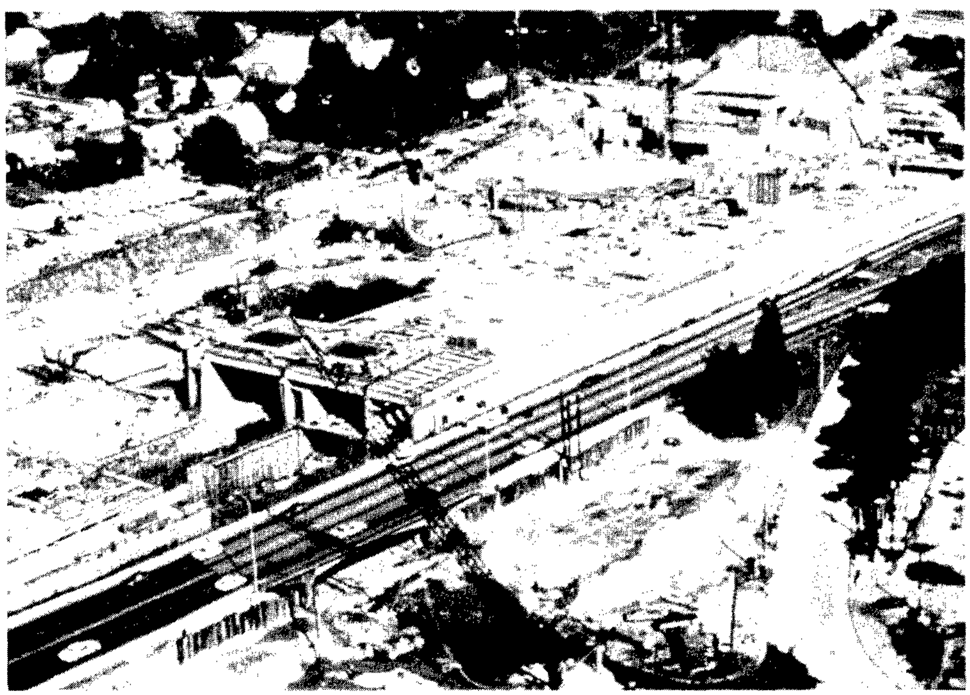


Figure 11. Vibro-compaction on Mt. Baker Ridge's Interstate 90.

6.2 AREA TREATMENT

Wando Terminal, Charleston, SC.[7]

In South Carolina, a site improvement challenge involved the expansion of Wando Terminal, a State port facility in Mount Pleasant near Charleston. The expanded terminal is designed to serve as a docking facility and as a 225,000 m^2 concrete-paved container storage yard. The site of the expansion section is located north of the existing facility (figure 12). The storage yard for the expansion is divided into three areas: Area A (44,500 m^2), Area B (72,900 m^2), and Area C (109,300 m^2). Areas A and B were formerly marshlands that, over 10 years ago, had been filled to elevation +6.7 m MLW or higher. The long-term surcharging of these areas had consolidated the underlying marsh deposit sufficiently to eliminate the need for additional ground improvement. However, Area C was composed of virgin marshlands.

Much of the Charleston peninsula is composed of former marshland, filled over the last 350 years with both earthen materials and man-made debris. Many different structures have been built within these areas, and numerous problems have resulted, including areal subsidence in the range of 50 to 100 mm per year over the life of a structure. In the early design stages for the new container storage yard, the owner decided that the above settlements could not be tolerated. Since the existing Wando terminal is viewed as the showcase of South Carolina State Ports Authority's Charleston facilities, the expansion was required to be of comparable quality. Replacing some, or all, of the deep deposits of marsh mud in Area C with less compressible soil was determined to be the only option.

A generalized profile of the subsurface conditions within Area C is depicted in figure 13. This profile essentially represented the worst-case conditions for analyzing the various ground improvement alternatives being considered to create the container storage yard. As can be inferred from the soil properties listed in figure 14, the marsh mud was extremely soft and compressible. Although still relatively soft, the intermediate "firm" clay was more consistent and did not present the same design challenges with respect to compressibility and stability. The lower stratum (Cooper Marl), due to its high over-consolidation ratio, is virtually incompressible.

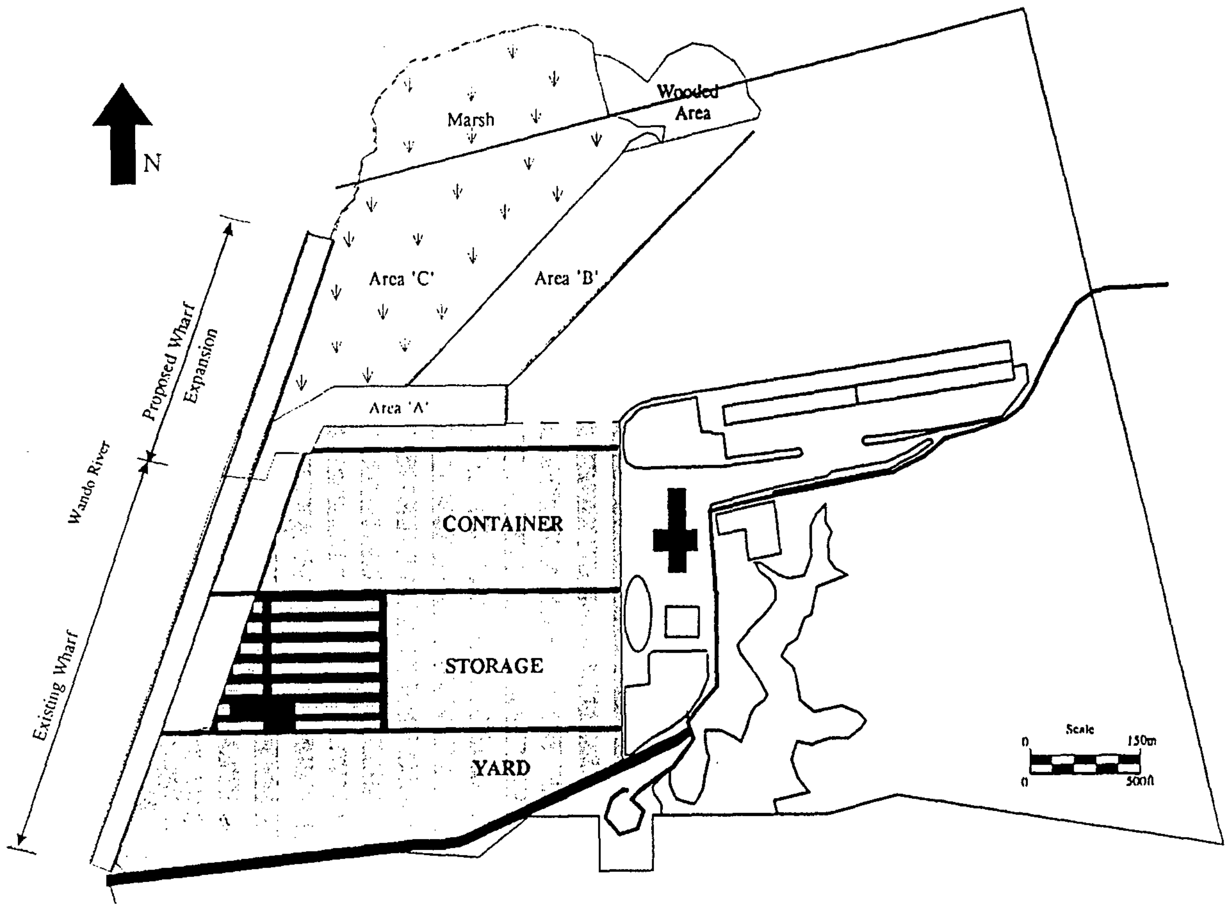


Figure 12. State Pier 41, Wando Terminal

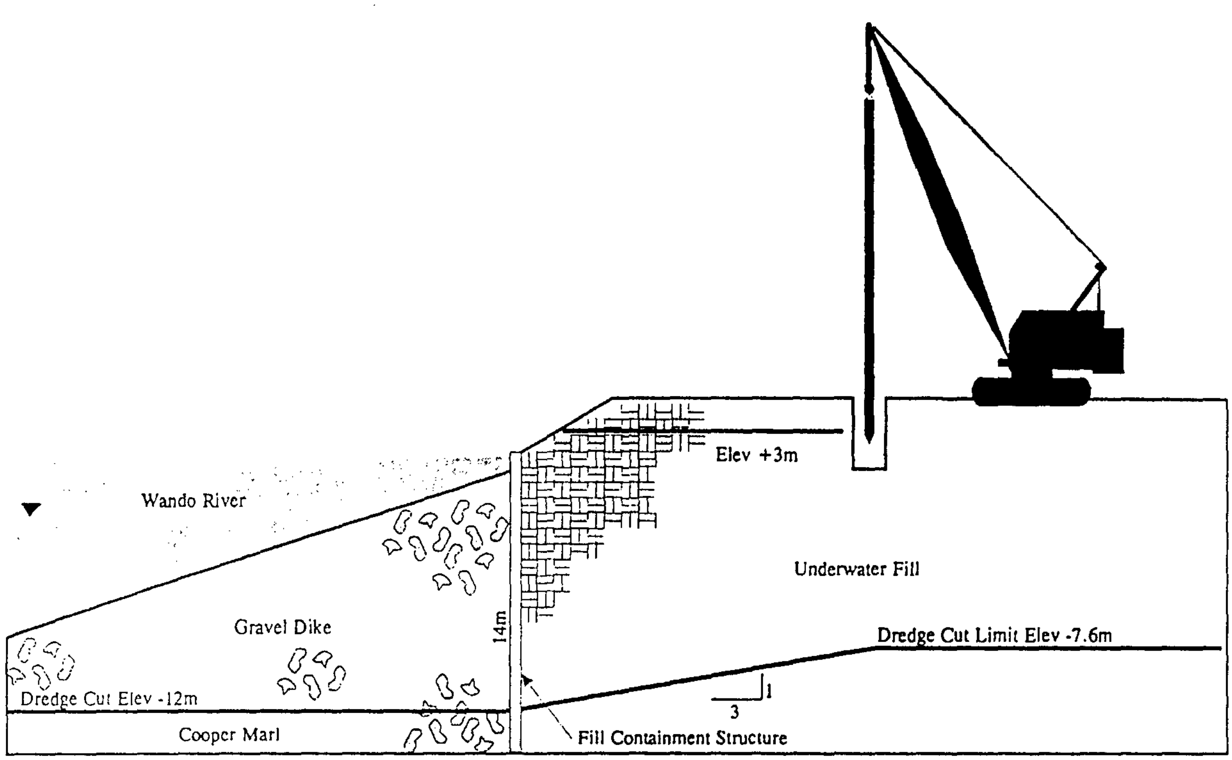


Figure 13. Vibro-compaction at Wando Terminal.

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

1) Dredging the soft clay to elevation -7.5 m and replacing that material with 1.2 million cubic meters of clean sand to elevation +3 m.

2) Installing vertical drains (wick drains) to accelerate the consolidation of the underlying clays.

3) Transforming the very loose sand backfill into dense sand using vibro-compaction (figure 14).

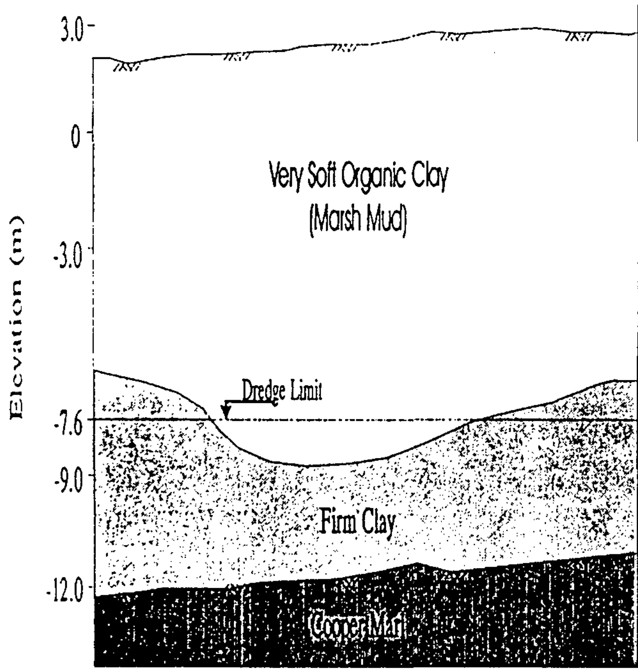
The third step, involving vibro-compaction, would densify the backfill and eliminate a costly intermediate step of dewatering the site.

By selecting a dredge level of -7.5 m MLW, only isolated pockets of the highly compressible marsh mud would be left in place. By backfilling with clean sand (fines $\leq$ 5 percent), and inserting wick drains to elevation -12.2 m MLW on 1.5 m centers, it was estimated that the maximum post-construction settlement of the container yard would include 90 to 180 mm of primary consolidation over the first 2 years and up to 100 mm of secondary compression over the next 50 years.

Once the specifics of the program were determined, the backfilling of the 109,300-m^2 excavation with underwater fill could begin. This fill was specified to be a fine sand with less than 1 percent clay and less than 5 percent fines (silt and clay) by weight. The contractor elected to fill the excavation by hydraulically pumping the sand from a central dumping area.

With the backfill and wick drains in place, vibro-compaction (utilizing 4 rigs working double shifts, 6 days per week for 5 months) could complete the improvement program (figure 15). At the completion of this process, the sand backfill was completely densified, lowering the surface elevation from 10.7 m to 9.5 m.

Pre-treatment and post-treatment cone penetrometer test results (figure 16) confirmed that the required minimum densification had been achieved. Vibro-compaction densification stabilized the soils to support the weight of the containers. Additionally, since the Wando Terminal site is situated within one of the most prominent areas of seismicity along the Atlantic Seaboard, the densification served as a precaution designed to prevent liquefaction of soils should an earthquake occur (Charleston was struck by a large earthquake in 1886).



SPT N-Value (b/0.3m)	Vane Shear Strength (kPa)	Density (kg/m^3)	Natural Water Content (%)	void Ratio	Compressibility Ratio ($C_c/1+e$)
<1	3.3 to 9.6	500 to 865	95 to 191	2.5 to 3.1	0.23 to 0.58
4 to 9	<24	1300 to 1490	<88	0.89 to 1.80	0.09 to 0.2
5 to 37	>150	1680 to 1842	35 to 65	--	OCR = 3 to 6

Figure 14. Generalized subsurface profile of Area C.

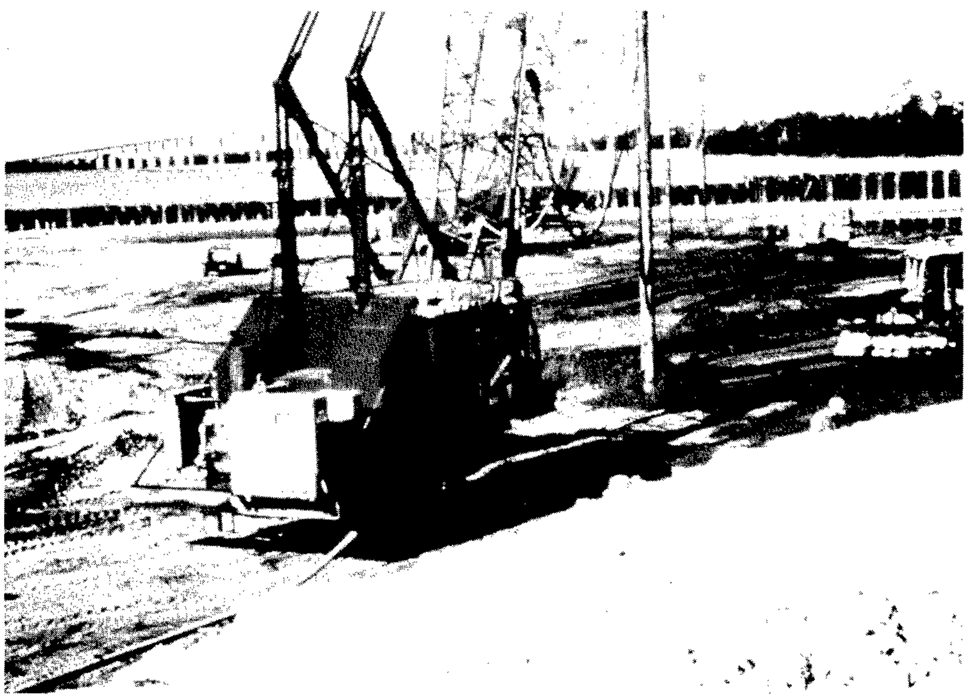


Figure 15. Densification of loose sand backfill using vibro-compaction at Wando Terminal.

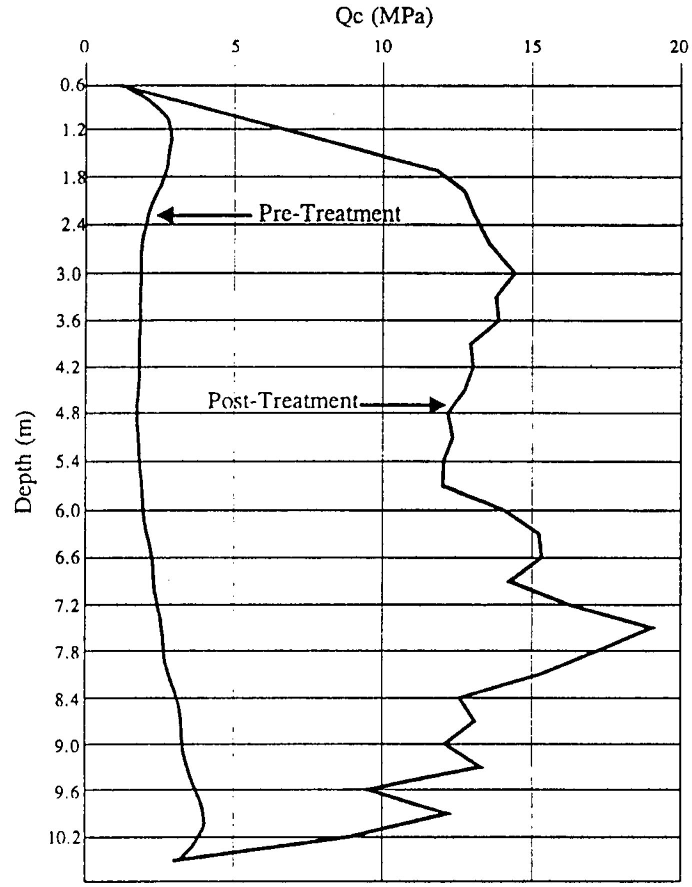


Figure 16. Sample Cone Penetrometer Test results.

The uniqueness of this large, multi-step project was twofold. First, its size (109,300 m^2) proved to be one of the largest areas treated to date. Second, the program achieved project goals by backfilling the excavation through the water instead of traditional dewatering and filling the hole with compacted layers of sand. Vibro-compaction proved to be equally effective and considerably more economical than the dewatering alternative.

6.3 DENSIFICATION OF GRANULAR BACKFILL

Port of Kismayo, Somalia[4]

The Port of Kismayo is located 45 km below the Equator on the coast of the East African country of Somalia. The port facilities at Kismayo include a four-berth wharf with a total length of 630 m. Originally constructed in the mid-1960s, the wharf consisted of a 18.5 m wide precast concrete platform supported by precast, prestressed concrete piles. Soon after completion of the structure, serious deterioration was observed. As a result, a complete rehabilitation of the port was implemented from 1986 to 1988.

As shown in figure 17, the replacement structure includes a 14.0-m-high anchored sheet pile bulkhead located outboard of the existing wharf. To build this, the existing wharf platform was demolished and the tops of the existing concrete piles were cut off. A steel sheet pile bulkhead was then installed 11.0 m outboard of the existing platform, and supported by a continuous steel sheet pile deadman located near the back of the platform. Behind the bulkhead, a granular backfill was placed underwater, then compacted from the surface using deep vibratory compaction. Deep compaction was used to minimize settlement of the backfill, particularly differential settlement in the vicinity of the cut-off piles where wide variability in the density of the loosely placed backfill was anticipated. The use of vibro-compaction would, therefore, minimize the need for future maintenance of the wharf's rigid concrete pavement and avoid possible disruption to the surface drainage system. Vibro-compaction was also used to increase the passive soil resistance for support of the deadman anchorage.

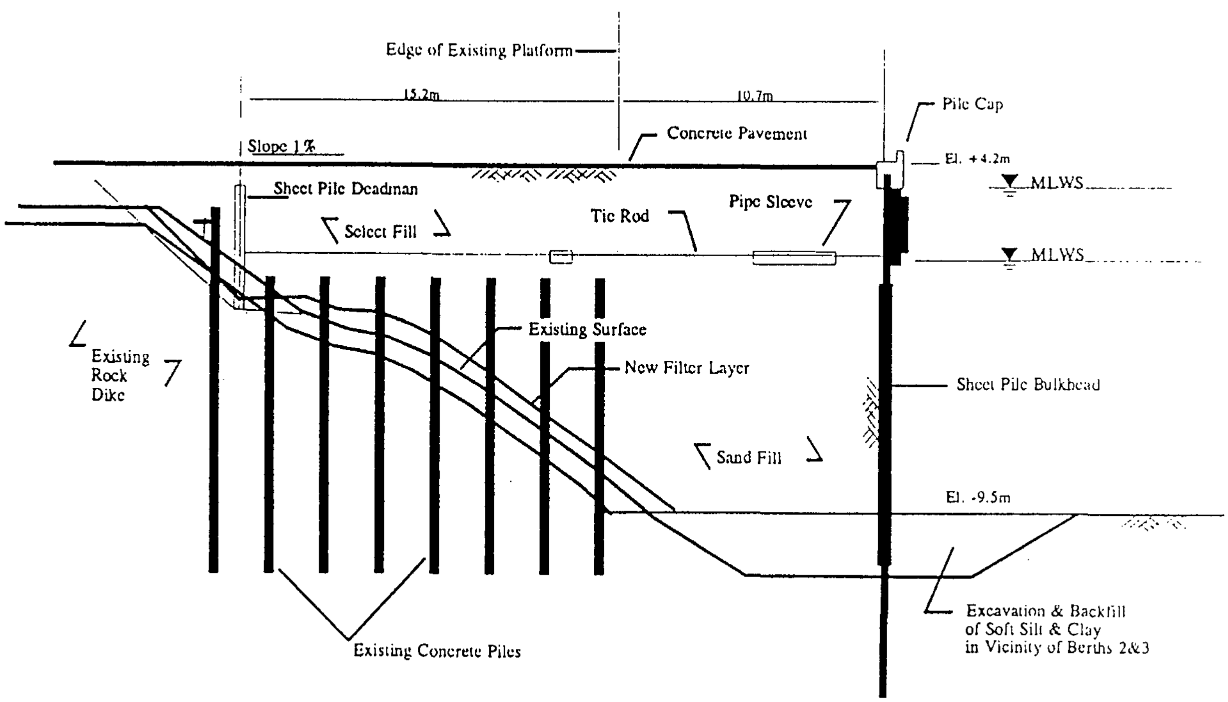


Figure 17. Typical section of anchored sheet pile bulkhead-Port of Kismayo.

Two types of underwater fill material were specified, including sand fill behind the bulkhead and a select fill at the deadman for increased passive soil resistance. The sand fill used in construction consisted of a uniformly graded, medium-to-fine beach sand. The select fill was a sand-gravel mix with about 30 percent gravel size. The maximum height of the sand fill above the sea bottom was about 14 m. The select fill had a depth of about 5.5 m.

The construction specifications required that the backfill material placed below water be compacted to a minimum relative density of 80 percent as determined by in-situ testing and the specified correlations described below. The contractor was given the freedom to select the appropriate equipment, subject to the approval of the engineer and verification by an initial field testing program. In-situ quality control tests performed during the initial field testing program included both static cone penetrometer tests (CPT) and standard penetration tests (SPT).

To prevent permanent sag of the bulkhead tie-rods, the vibro-compaction operations in the sand fill area were required to be performed in two stages. The first stage compaction, performed when the fill reached the tie-rod level, extended to the bottom of the fill. After completion of Stage I compaction and correction of any resulting sag of the tie-rods, the remaining fill was placed and Stage II compaction performed only to the tie-rod level.

The spacing of the compaction points was determined from the initial field tests. However, to avoid any interference with the tie-rods, the compaction point spacing parallel to the face of the bulkhead was set equal to the tie-rod spacing of 2.0 m.

Prior to production, various types of equipment, compaction procedures, and backfill material were evaluated. The most successful results were obtained using two types of vibrators--a hydraulically operated one that utilized water jetting and gravel backfill material and an electrically operated one that utilized water jetting and select backfill material. Both pieces of equipment produced the required densification for the full depth of fill. With these successful test results, the contractor incorporated these pieces and compaction methods into the actual program.

With the project equipment selected, five initial test trials were performed to assess Stage I compaction of sand fill near the wharf bulkhead. No backfill material was introduced during the operations and deep compaction in the sand fill area was positioned outboard of the bulkhead. Of all the trials performed, only two yielded acceptable results. The fourth trial utilized a single electrically operated vibrator that penetrated to a depth of 11.7 m, and the fifth trial utilized a double electrically operated vibrator that penetrated to 12.0 m. Both trials reached the specified minimum density of 80 percent.

In addition to CPT testing, the performance of Stage I deep compaction in the sand fill area was also assessed by determining the volume reduction of the fill. Since no backfill material was added to the probe holes during compaction operations, it was possible to estimate the volume reduction from ground surface settlement measurements. Settlements were determined by optical survey of the ground surface for a grid of points within the test area.

It was found that the twin electrically operated vibrators, with twice the energy of the single type, resulted in approximately 44 percent greater volume reduction of the fill. These results, as well as the adequate CPT results, illustrate the greater densification that can be achieved using twin vibrators. In addition, the use of twin vibrators can cut the time for compaction work almost in half.

Concern was raised during design that deep compaction of the backfill may result in a significant increase in the lateral earth pressures acting on the bulkhead. Rather than increase the strength of the bulkhead and its anchorage to resist the additional pressures, the construction specifications stipulated that no compaction be done immediately behind the bulkhead for a distance to be determined from field testing (this was done using CPT tests).

Except for a dense zone from elevation -3.0 to -4.5 m in the pre-compaction CPT, there was little difference between the cone resistance values obtained prior to compaction and the post-compaction values for CPT-10 located 4.0 m beyond the last row of compaction probes. Using CPT cone resistance values as an index of in-situ strain and stress conditions, it was concluded that beyond a distance of about 4.0 m there was little or no increase in lateral ground stresses developed as a result of vibratory deep compaction operations. With these results, the first row of compaction probes were placed at a distance of 4.0 m from the centerline of the bulkhead.

This project illustrates the successful use of vibratory deep compaction to density loose, underwater fill behind a marine bulkhead. This technique resulted in significant compaction of the fill without impacting the bulkhead or its anchorage system. As a result of this work, it is anticipated that post-construction settlement and related maintenance costs will be substantially reduced. Namely, it was realized that compaction of clean, uniform sized underwater fill can be successfully performed without adding fill material to the probe hole. Also, twin vibrators, operated simultaneously, provide greater soil densification and significantly shorten the duration of the deep compaction work. However, for future projects utilizing vibratory deep compaction adjacent to steel sheet pile bulkheads, consideration should be given to the use of strain gages and inclinometer casing attached to the sheet piling to determine stresses and deflections of the bulkhead with depth. With these data, the estimation of the corresponding lateral earth pressures can be determined.

6.4 LIQUEFACTION MITIGATION

Manchester Airport, New Hampshire[18]

Construction of a new, 15,000 square meter terminal building at Manchester Airport, New Hampshire over loose, sandy, potentially liquefiable soils required that a ground improvement program be developed to mitigate the risk of liquefaction during a seismic event.

Design phase borings had revealed delta-deposited, clean, uniformly-graded, saturated, fine-to-medium sands from depths of 3.7 to 13.7 meters. Laboratory gradation and Standard Penetration Testing revealed the potential for seismically-induced liquefaction. The design of the densification program was based on specific parameters developed from:

1) Methodology proposed to determine the factor of safety against the occurrence of liquefaction.[20]
2) Correlation of SPT values to volumetric strain.[22]

Analysis performed in accordance with the above indicated a factor of safety against liquefaction of less than unity under regional design criteria and a volumetric strain of ten percent of the layer thickness that translated into a potential for 0.3 m of settlement below the building footprint.

Both deep foundations (piles, drilled piers, pressure injected footings) and ground improvement alternatives to allow shallow footing construction were evaluated. The deep foundation alternatives were eliminated due to cost considerations and the uncertainty of performance under liquefaction conditions. Of the ground improvement alternatives considered (that also included excavation/replacement, dynamic compaction, deep blasting, and compaction grouting) vibro-compaction was selected because of its cost-effectiveness and proven success record in sands.

The vibro-compaction design was required to meet seismic criteria of a design earthquake magnitude of 6.0, a peak ground acceleration of 0.12g, and a minimum factor of safety against liquefaction of 2.0. Allowable differential settlement was determined to be 12 mm, with a 25-mm allowable total settlement.

To meet this criteria, compaction points were located on a 3 m by 3 m grid. The necessary depths of compaction were determined to be 8 meters and 11 meters. Although design borings had identified potentially liquefiable soils to 13.7 m, actual depth-of-treatment selection was based upon performance studies of Japanese sites where liquefaction had occurred.

The spacing and depth of treatment used resulted in the minimum specified relative density where coarse, clean sand was present. Where post-treatment tests indicated that loose relative density conditions remained, the spacing was reduced.

The project required over 2,600 compaction points. Thirty four post-treatment SPTs were conducted, typically at 900 square meter intervals at the centroid of the compaction point grid to assess the vibro program. Compliance with project specifications was generally achieved after initial treatment. Where SPT values at or slightly below specified values were found, at depths ranging from 3.7 m to 5.2 m, this was attributed to the presence of a dense crust of coarse sand temporarily arching over the loose material below. Subsequent testing, after a waiting period of 1 to 3 weeks, showed that, in most instances, N values had increased with time to meet the specified criteria.

On this project, practical methods were used to assess the potential for the occurrence of liquefaction and to develop project specifications for risk mitigation. To date, few projects have been reported in which field data have been analyzed using the methodology described, and then designed, constructed, and verified to the specific criteria.

CHAPTER 7

COST DATA

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

7.1 BUDGET ESTIMATE

Using the criteria described in chapter 4, vibro-compaction point spacing can be determined. The total area requiring improvement can then be divided by the effective area of each point to determine the number of vibro-compaction locations required. In estimating costs, it is important to include the perimeter zone outside the limits of loaded area or influenced by vibro-compaction in the surface area calculation so that the project requirements are accurately matched. The depth of improvement required can then be multiplied by the number of points to determine budget footage of vibro-compaction. It is normally more economical to lower the entire site elevation by the vibratory compaction effort rather than add granular backfill from the surface. A typical price per linear meter of vibro-compaction would be $15 when no backfill is placed around the probe and $25 when granular backfill is added. The specific backfill cost will vary significantly on a local basis. In addition, a mobilization/demobilization cost of $15,000 per rig should be added. Other costs that should be considered are:

- Surface densification--with the lack of overburden restraint, the upper meter of soil will have to be densified by conventional surface compaction methods.

- Additional fill to raise the site to the required grade and, in the case of no added backfill, to compensate for the site's depression. This cost will depend on the looseness of the in-situ soil and the specified degree of densification.

- Verification testing--Standard Penetration Tests (SPT) are normally specified and, to ensure uniformity, some tests should be continuous. On large projects, SPTs are commonly supplemented by Cone Penetrometer Tests (CPT). Both of these tests are performed at the centroid of the vibro-compaction points, thus giving the lowest readings.

Costs

For typical area densification problems, the range for a vibro-compaction solution will vary from $1.00 to $4.00 per cubic meter of densified in-situ soils, depending mainly on the size of the project, gradation of in-situ soils, and degree of densification required. However, in marginal soils where special backfill is required, the costs could be significantly higher, yet the total economics may justify a vibro-compaction solution.

Table 5 is a listing of the many factors that can affect the pricing of a vibro-compaction project.

Table 5. Factors affecting price of Vibro-compaction projects

In-situ Material	•Type of Material •In-situ Density •In-situ Cementation
Backfill Material	•Type •Cost
Densification Requirements	•Load Bearing •Degree of Densification 1) Average relative density 2) Minimum relative density
Project Requirements	•Size •Depth of Densification •Overburden •Type 1) Footing compaction 2) Area compaction •Specifications •Location of Project 1) Labor and union considerations 2) Support equipment availability 3) Weather - freezing weather conditions
Pricing	•Compaction Spacings •Unit Pricing 1) Linear meter 2) Cubic meter

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

1. Determine the performance requirement. Chapter 4 lists typical requirements for most projects.

2. Determine the number of compaction points required from the performance requirement, resulting compaction point spacing, and total project size.

3. Determine the required depth of compaction from the subsurface investigation and project requirements.

4. Cost of vibro-compaction = (# of compactions x depth x unit price) + mobilization.

5. Price includes supervision, labor, equipment, tools, utilities and backfill added during compaction.

6 Rate of production = 100 linear meters per vibrator per 8 hour day.

7. About 0.8 cubic meter of backfill added for each 1.5 linear meters of compaction.

7.2 SUMMARY OF TYPICAL PROJECTS

Table 6 is a limited summary of transportation related Vibro Compaction projects indicating key features and costs.

Table 6. Summary of Typical Projects.

Project	Date	Remarks	Contract Value
Foundations for Elevated Highways Quebec City, PQ	1971	Vibro-compaction treated the ground for the foundations of 6 sections of elevated highway connecting Parliament Hill with the lower town of Quebec City.	$400,000
Palm Beach Harbor Port Everglades, Fl	1990	Vibro-compaction to compact up to 16.8 m deep backfill behind a bulkhead wall measuring approximately 306,000 m^3.	$423,000
Highway 100 Minneapolis, MI	1991	Vibro-compaction beneath a series of parallel retaining walls.	$150,000
North Parking Structure Grand Rapids, MI	1990	Vibro-compaction to 480 kPa for foundation loads of up to 8,900 kN. Utilized 2 types of vibrators to work up to within 1.5 m of retaining wall system and utilities.	$112,000
Manchester Airport Manchester, NH	1992	Vibro-compaction of 7.6 to 12.1 m deep, loose, silty sand deposit in preparation for construction of new airport terminal building.	$681,000
Bridge Foundations South Roxna, IL	1994	For Illinois DOT, 130 vibro-compaction points under 2 bridge piers to 10.7 m depth for liquefaction risk mitigation.	$115,000

REFERENCES

1. Vibroflotation Foundation Co. Literature, March, 1980. 8 pages.

2. M.P. Moseley, Ed (1993), "Ground Improvement," *Blackie Academic & Professional*, Boca Raton, 218 pages.

3. Brown, R.E., (1977), "Vibroflotation Compaction of Cohesionless Soils," *Journal of the Geotechnical Engineering Division, Vol. 103, No. GT12,* pp 1437-1451.

4. D'Appolonia, E., Miller, L.E., and Ware, T.M. (1953), "Sand Compaction by Vibroflotation." *Proceedings, ASCE, Vol. 79, Separate No. 200,* pp 1-23.

5. Jebe, W., and Bartels, K., (1983), "The Development of Compaction Methods with Vibrators from 1976 to 1982." *Proceedings of the Eighth European Conference on Soil and Foundation Engineering*, Helsinki, Vol. 1, pp. 259-266.

6. Castelli, R.J. (1991), "Vibratory Deep Compaction of Underwater Fill." Proceedings of Deep Foundation Improvements: Design, Construction, and Testing. *ASTM STP 1089, Melvin I. Esrig and Robert C. Bachus, Editors.* ASTM, Philadelphia, PA. pp 279-296.

7. Hussin, J.D. and Foshee, F. (1994), "Wando Terminal Ground Improvement Program." *Proceedings of Dredging '94, ASCE*, New York, NY. pp 1416-1425.

8. Task Force 27 Report (1990), "In-situ Soil Improvement Technologies." Prepared for *AASHTO-AGC-ARTBA Task Force 27 Ground Modification Techniques*, Washington, D.C., 324 pages.

9. Mitchell, J.K. (1970), "In-Place Treatment of Foundation Soils." *Journal of the Soil Mechanics and Foundations Division*, Vol 96, No. SM1, pp. 73-110.

10. Mitchell, J.K. (1981), "Soil Improvement: State-of-the-Art Report." *Proceedings of the Tenth International Conference on Soil Mechanics and Foundations Engineering*, Stockholm, Sweden, Vol. 4. pp. 509-565.

11. Welsh, J.P. (1987), "Soil Improvement - A Ten Year Update." *ASCE Geotechnical Special Publication No. 12*, 331 pages.

12. Xanthakos, P.P., Abramson, L.W., and Bruce, D.A. (1994), "Ground Control and Improvement." *Wiley-Interscience Publication, John Wiley and Sons, Inc.*, New York, NY. 910 pages.

13. Rollins, K.M. Editor (1994), "In-Situ Deep Soil Improvement," *Geotechnical Special Publication No. 45, ASCE*, New York, NY. 149 pages.

14. Hryciw, R.D. Editor (1995), "Soil Improvement for Earthquake Hazard Mitigation," *Geotechnical Special Publication No. 49*, ASCE, New York, NY. 141 pages.

15. Schaefer, V.R. Editor (1997), "Ground Improvement, Ground Reinforcement, Ground Treatment Developments 1987-1997," *Proceedings of the Geo-Institute, Geotechnical Special Publication No. 69*, ASCE, New York, NY. 616 pages.

16. Chellis, R.D. (1951), "Pile Foundations." *McGraw Hill Book Company*, New York, N.Y., pp 25-251.

17. Kimmerling, R. E., (1994), "Blast Densification for Mitigation of Dynamic Settlement And Liquefaction," *Washington DOT Report No. WA-RD 348.1.*

18. Sobel, J.M. et al., (1993), "Liquefaction Risk Management - Manchester Airport," *Proceedings of the Third International Conference On Case Histories In Geotechnical Engineering*, St. Louis, Mo, pp 1709 -1713.

19. Mitchell, J.K., and Solymar, Z.V. (1984), "Time Dependent Strength Gain in Freshly Deposited or Densified Sand." *Journal of Geotechnical Engineering. Vol. 110, No. 10, ASCE GT.* pp. 1415-1430.

20. Seed, H.B. et al. (1985), "Influence of SPT Procedures in Soil Liquefaction Resistance Evaluations," *Journal of Geotechnical Engineering, ASCE, Vol. 101, No. 12*, pp 1245-1445.

21. __________ (1989), "I-90 Lid Structure - Mt. Baker Ridge, Seattle, Washington," *Vibro Systems Case Histories, GKN Hayward Baker*, 4 pages.

22. Tokimatsu, K. and H.B. Seed, (1987), 'Evaluation of Settlements in Sands Due to Earthquake Shaking," *Journal of Geotechnical Engineering, ASCE, Vol. 113, No. 8.* pp 861-878.

DYNAMIC COMPACTION

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CHAPTER 1

DESCRIPTION AND HISTORY

1.1 DESCRIPTION

Dynamic compaction is a method of ground improvement that results from the application of high levels of energy at the ground surface. The energy is applied by repeatedly raising and dropping a tamper with a mass ranging from 5 to 18 Mg at distances ranging from 9 to 30 m. The tamper is lifted and dropped by a conventional crane with a single cable plus a winch that has a free spool attachment that allows the single cable to unwind with minimum friction. The tampers energy of impact at the ground surface results in densification of the deposit to depths that are proportional to the energy applied. The depth of improvement generally ranges from about 3 to 11 m for light to heavy energy applications, respectively. Following the high energy level application, the surface of the deposit is in a loose condition to a depth equal to the depth of the craters. This surface layer is sometimes compacted on a tight grid basis with a low level energy application called an ironing pass.

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

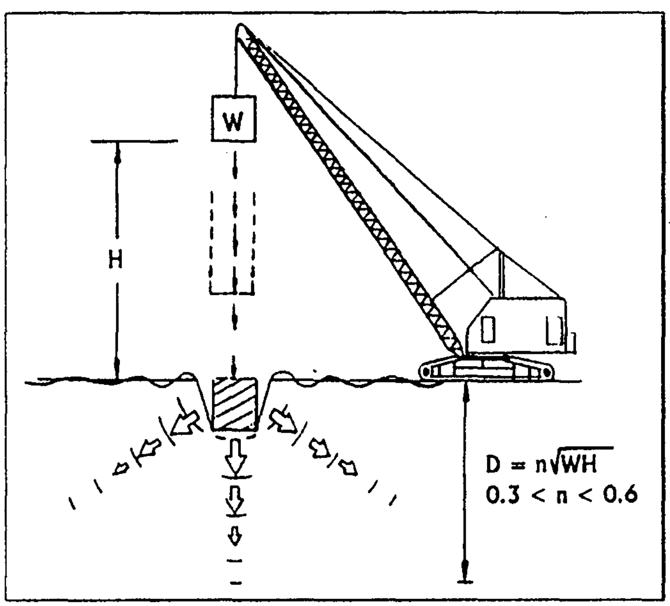

Figure 1. Schematic illustration of dynamic compaction.

If ground improvement is needed to provide a suitable bearing stratum for an embankment or a structure, dynamic compaction may be indicated because:

- Dynamic compaction is generally the most economical form of site improvement. Conventional cranes are used to lift and drop the tampers, which allows local contractors to compete with specialty contractors to perform this work.

- Dynamic compaction can be used on a wide variety of deposits, ranging from silty soils to boulder-sized granular deposits. This procedure can improve both fill deposits and natural soil deposits can be improved by this procedure.

1.2 HISTORICAL OVERVIEW

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

These examples of tamping to densify soil were used only for a specific project or a limited number of projects, and the technology was never developed further.

Use of dynamic compaction began on a regular and continuing basis in Europe in 1969 and in the United States in 1971. In Europe, tampers of 8 to 10 Mg were dropped from heights of 8 to 12 m to densify fill deposits. This process was called heavy tamping and was generally used in good quality fill deposits such as rock waste, rubble, and sand. After a few years, the process was expanded to include fine grain-soil deposits and the name was changed to dynamic consolidation.(6)

In the United States, densification was initially achieved using tampers in the range of 1.8 to 5.4 Mg with drops of 6.1 to 10.7 m to densify loose rubble fill and granular deposits to support lightly loaded structures. Later, tampers up to 15 Mg were used to densify former landfills. This technique was initially called "pounding," but eventually became know as dynamic compaction.(7)

Before 1975, European and American practices developed independently and were somewhat experimental. In 1975, a technical article was published that dealt with the theoretical and practical aspects of dynamic compaction.[6] A formula was given for predicting the depth of improvement as a function of the applied energy. This article presented sufficient information to place the dynamic compaction process on a sound technical basis. Subsequent articles published in Europe and the United States used this paper as the starting point to expand and exchange the knowledge base of dynamic compaction.

1.3 FOCUS AND SCOPE

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

- Where dynamic compaction is appropriate and inappropriate.
- Elementary design and construction concepts.
- Construction monitoring.
- Specifications.
- Typical case histories.

This technical summary does not provide an in-depth discussion of design and construction considerations for dynamic compactions. FHWA-SA-95-067, Dynamic Compaction provides more details regarding all of the factors that need to be considered for planning and implementing a dynamic compaction project.[8] The reader is referred to these manuals and to the other references contained in this document for more detailed information.

1.4 GLOSSARY

This glossary describes terminology unique to dynamic compaction.

Applied Energy — Average energy applied at ground surface, which is calculated on the basis of the sum of all the energy applied by dynamic compaction divided by the surface area of the densified soil. The typical units are Joules per meter squared (kJ/m^2).

Crater	Depression in the ground at the drop point location that results from energy application.
Depth of Improvement	Maximum depth to which measurable improvement is attained.
Drop Energy	Energy per blow, which is calculated on the basis of the tamper mass multiplied by the drop height.
High-Level Energy	Energy applied to cause densification to the depth of improvement.
Induced Settlement	Average ground settlement following densification, which is determined by elevation readings taken before and after dynamic compaction.
Low-Level Energy	Energy applied to compact the surface deposits to the depth of crater penetration following high-level energy application. Low-level energy application frequently is called the ironing pass.
Pass	The application of a portion of the planned energy at a single drop point location. Multiple drops are required to deliver the energy at each drop point. If all the drops cannot be applied at one time because of deep craters or excess pore water pressures, another pass or passes will be required after excess pore water pressures dissipate or the craters are filled with granular fill. There is generally a waiting period of at least a few days between passes.
Phases	Describes the pattern in which the energy will be applied. For example, every other drop point of the grid pattern could be selected to be densified as Phase 1. After completion of Phase 1, the intermediate drop points could be densified as Phase 2. Some projects use only one phase; others have been undertaken with as many as five phases.

CHAPTER 2

DESIGN CONSIDERATIONS

2.1 INTRODUCTION

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

- What are the typical applications of dynamic compaction?
- What are the advantages and disadvantages of using dynamic compaction?
- What types of deposits can be improved by dynamic compaction?
- Are there limitations or environmental considerations?
- Are there alternate site improvement methods that also should be considered in lieu of dynamic compaction?

2.2 APPLICATIONS

The primary purpose of dynamic compaction is to densify natural and fill deposits to improve soil properties that affect performance. The primary uses for which dynamic compaction has been used to date include:

- Densification of loose deposits.
- Collapse of large voids.

a. Densification of Loose Deposits

The primary use of dynamic compaction is to densify loose deposits so as to reduce the settlement under load application. Densification results from the soil grain particles being compacted to reduce the void ratio. In partially saturated soils, densification is similar to laboratory impact compaction by the proctor method. In saturated or nearly saturated fine sands and silts, excess pore water pressures develop on impact which dislodges some of the point-to-point contacts between soil particles. Following dissipation of the pore water pressures, the soil grains restructure into a denser state of packing at a lower water content.

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

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

Figure 2 illustrates the soil behavior during dynamic compaction. The soil deposit is characterized as a phase diagram consisting of three portions, i.e. solid materials, liquids, and gas. On the left side of figure 2, the phase diagram represents the original state of the soil deposit. During the first phase of energy application, excess pore water pressure develops in the liquid portion and then dissipates with time. The time required for pore pressure dissipation is a function of the permeability of the soil mass and the distance to a drainage path. There is some induced ground settlement which is represented by a lowering of the top of the phase diagram. This induced settlement results from expulsion of some of the gas and liquid for the voids which causes a lower void ratio. The solid portion remains the same.

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

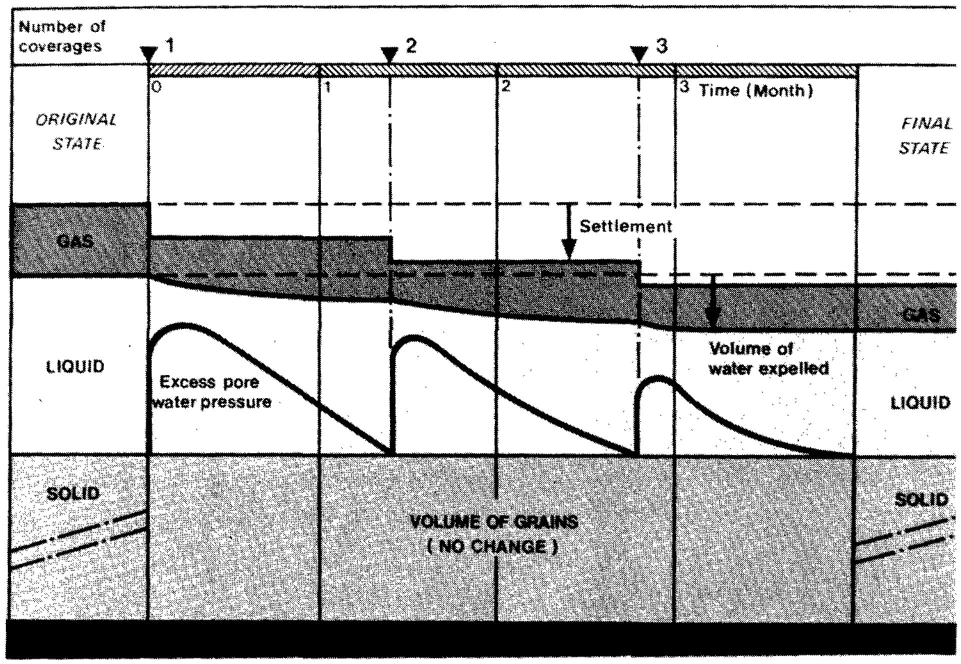

Figure 2. Phase diagram model of soil behavior during dynamic compaction (Courtesy of Soletanche.)

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

The final state of the soil mass is shown on the right side of figure 2. The volume of the solids remains the same as the initial state, but the volume of the liquids and the gas decreases. The reduced void ratio results in reduced compressibility and increased strength of the deposit.

Figure 2 was prepared for a condition where three phases of energy application would be made over the area. However, the principle is the same regardless of the number of phases.

b. Collapse of Large Voids

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

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

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

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

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

2.3 ADVANTAGES AND DISADVANTAGES

The advantages of dynamic compaction include:

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

- Densification of the deposits can be observed as the work proceeds. If multiple passes are made over an area, each succeeding pass generally will result in an average crater depth less that the prior pass. This is an indication of the improvement from the resistance of the ground. Ground settlement readings are usually taken before and after each application of energy, and the amount of ground compression is an indicator of the degree of improvement achieved. Normally, ground compressions of 5 to 10 percent of the thickness corresponding to the depth of treatment, (as predicted by equation 1 of chapter 4), occurs during densification. In extremely loose fills such as recent landfills, the ground compression can be 20 to 25 percent.

- Dynamic compaction can be used at sites with a very heterogeneous mixture of deposits and gradation ranges from large boulders and broken concrete to silty soil particles. Dynamic compaction is effective in densifying all of these deposits with the same equipment. Furthermore, deposits that were formerly thought uncompactable, i.e. building rubble debris or decomposed sanitary landfills, can be compacted by this method.

- Densification usually results in a bearing stratum having a more nearly uniform compressibility which minimizes differential settlements. Weaker zones within the deposit undergo the most improvement, which eliminates zones of potentially high compressibility.

- Densification can be achieved below the water table in pervious and semi-pervious deposits, which eliminates costly dewatering and/or lateral bracing systems required for conventional excavation and replacement techniques.

- Except for the very heavy tampers and the high drop heights, non specialty contractors can perform dynamic compaction on a local basis making the cost for dynamic compaction very competitive. For the larger tampers and the higher drop heights, the equipment must be modified because cable and drum wear is higher than normal. In this case, specialty contractors are required to perform the work.

- Dynamic compaction can proceed during inclement weather conditions including freezing weather or rain, provided precautions are taken to minimize water accumulation and frost penetration. Excess surface water should be removed by sloping the ground to shed water or by pumping. Deep frost penetration in fine grain deposits should be prevented by covering the surface with soil or straw removed immediately prior to dynamic compaction.

Disadvantages of dynamic compaction include:

- Dynamic compaction produces ground vibrations that can travel significant distances from the point of impact. In urban areas, this may require the use of lightweight tampers and low drop heights and limiting dynamic compaction to areas well within the property lines. At some sites, shallow trenches have been cut through the upper portion of the soil mass to reduce the transmission of energy off site.

- To prevent surface softening of the soil mass as well as sticking of the tamper into the soil, the ground water table should be located more than 2 m below ground surface. If the water table is higher than 2 m, it may be necessary either to lower the water table by pumping or to raise the grade by soil placement.

- At sites consisting of very loose deposits, such as recent landfills, it is frequently necessary to place a layer of granular material such as gravel or crushed stone at the surface to provide a working platform for equipment operation and to limit tamper penetration at impact. The surface layer also provides confinement for the underlying weak deposits. The cost of the granular fill can significantly add to the cost of the dynamic compaction operation.

- Lateral ground displacements of 25 to 76 mm have been measured at distances of about 6 m from the point of impact of 15- to 30-Mg tampers. Utilities or buried vessels within the zone of influence could be displaced or damaged.

2.4 SUITABILITY OF DYNAMIC COMPACTION FOR VARIOUS DEPOSITS

Soil parameters requiring evaluation to determine the suitability of dynamic compaction include:

- Permeability of the soil mass, which can be inferred from soil classification.

- Degree of saturation which is related to the position of the water table.
- Length of drainage path.
- Soil stratigraphy, such as buried hard or weak layers.

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

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

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

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

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

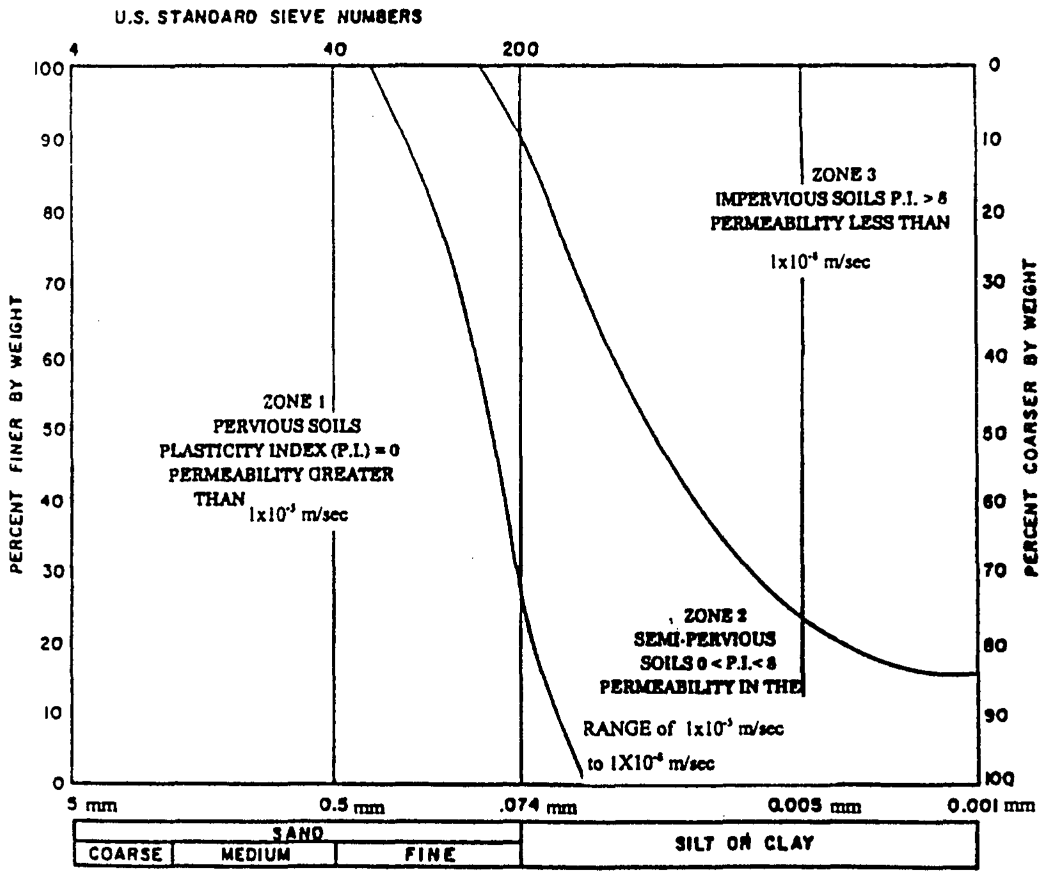

Figure 3. Grouping of soils for dynamic compaction.

- Zone 1 represents pervious soil deposits where dynamic compaction is best suited. The permeability of these deposits is generally greater that 10^{-5} m/s. Improvements are obtained regardless of whether these deposits are partially or fully saturated.

- Zone 3 represents impervious clayey deposits. The permeability of these soils is generally less than 10^{-8} m/s. Dynamic compaction is not recommended if these deposits are saturated or nearly saturated. Some improvement has been achieved in clayey fill deposits that are only partially saturated, provided the water content at the time of densification is at or below the plastic limit.

- Zone 2 represents intermediate soil deposit where improvements can be achieved provided the energy is applied in multiple phases to allow complete pore pressure dissipation between phases. The permeability of these deposits is generally between 10^{-5} to 10^{-8} m/s.

The suggested procedure for evaluating whether the soil deposits at any site are suitable for dynamic compaction is to first determine the grain size distribution of representative samples to determine where the gradation plots on figure 3. If the gradation plots in zone 1, proceed with the design and construction sequence for dynamic compaction. If the soils are in the range of zone 2 or 3, additional testing including water content and specific gravity will help to determine the degree of saturation. It would also be necessary to determine the position of the water table and to establish whether any pervious seams are present within the soil deposit. Additional field permeability testing should be considered as field permeability rather than the grain size gradation, dictates the suitability of the soil. If the testing confirms that the deposit is in zone 3, consider alternate methods of ground improvement. If the deposits are classified as zone 2, dynamic compaction will work, but the energy should be applied in phases.

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

2.5 LIMITATIONS AND ENVIRONMENTAL CONSIDERATIONS

Whenever a tamper impacts the ground, vibrations are transmitted through these deposits with diminishing intensity as the distance from the point of impact increases. If dynamic compaction is undertaken close to property lines, consider ground vibrations transmitted off site into adjacent facilities so as to prevent or reduce damage.

The U.S. Bureau of Mines has found that building damage is related to particle velocity.[9] Figure 4 was developed by the Bureau based on their experiences with damage measurements made in residential construction from blast-induced vibrations. The limiting particle velocity depends upon the frequency of the wave form. Normally, dynamic compaction results in frequencies of 5 to 12 Hz. Using figure 4 as a guide, this would limit peak particle velocities to values of 13 mm/s for older residences with plaster walls and 19 mm/s for more modern construction with drywall. Peak particle velocities that exceed the values given in figure 4 do not mean that damage will occur. Rather, these values are the lower threshold beyond which cracking of the plaster or drywall may occur.

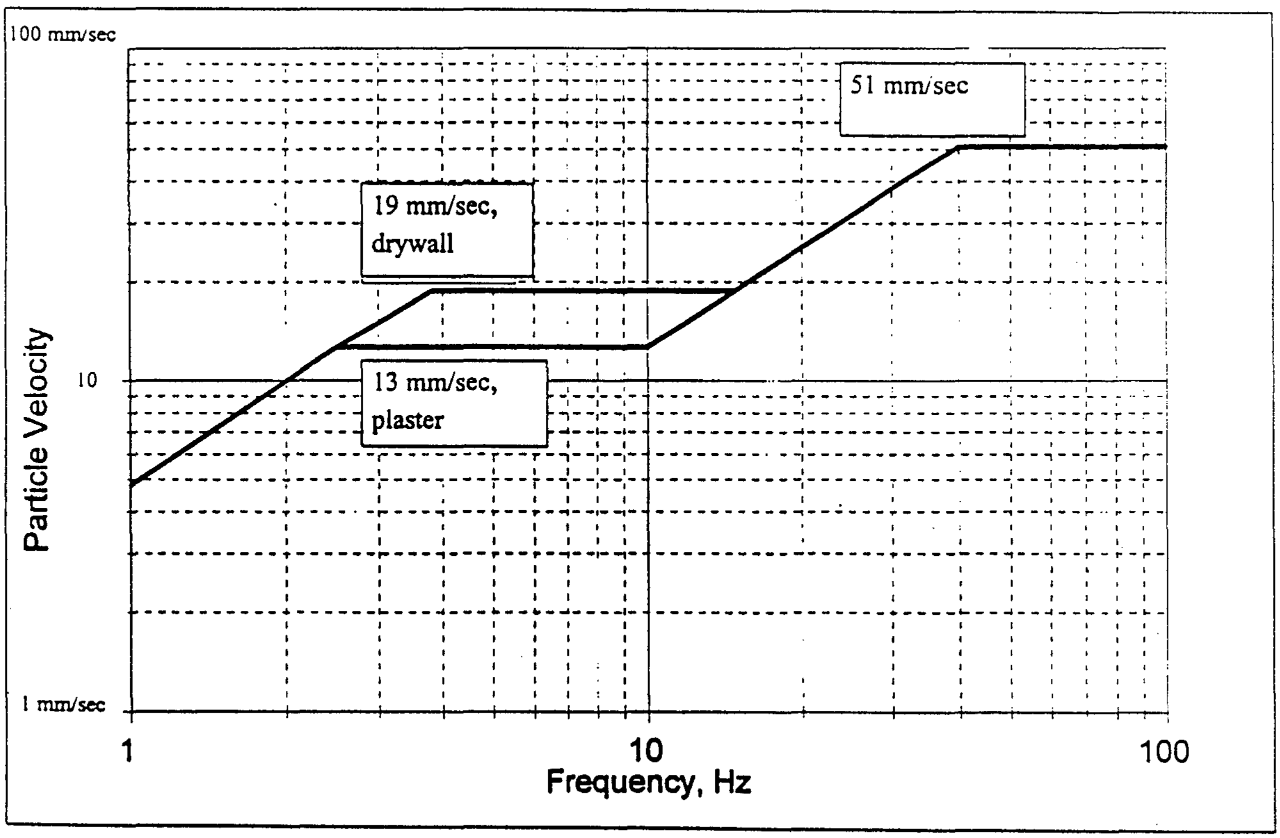

Figure 4. U.S. Bureau of Mines safe level of blasting vibrations for houses.

Data generated by the U.S. Bureau of Mines indicate that minor damage occurs when the particle velocity exceeds 51 mm/s, and major damage occurs when the particle velocity exceeds about 193 mm/s. Thus, keeping the particle velocity less that about 13 to 19 mm/s should be a reasonably conservative value to minimize damage.

Normally, the ground vibrations are measured with a seismograph at the time of construction. The readings are taken on the ground adjacent to nearby structures. However, before starting dynamic compaction operations, it is necessary to predict the velocity of ground vibrations because this may affect the level of energy application in close proximity to existing facilities. For planning purposes, figure 5 can be used. The square root of the energy of a single drop (drop height times the mass of tamper divided by the distance from the point of impact) is used to calculate the scaled energy factor. This value and the type of soil deposit that most closely resembles the soil being densified can then be used to predict the particle velocity. Figure 5 has been prepared on the basis of actual readings obtained at specific sites.

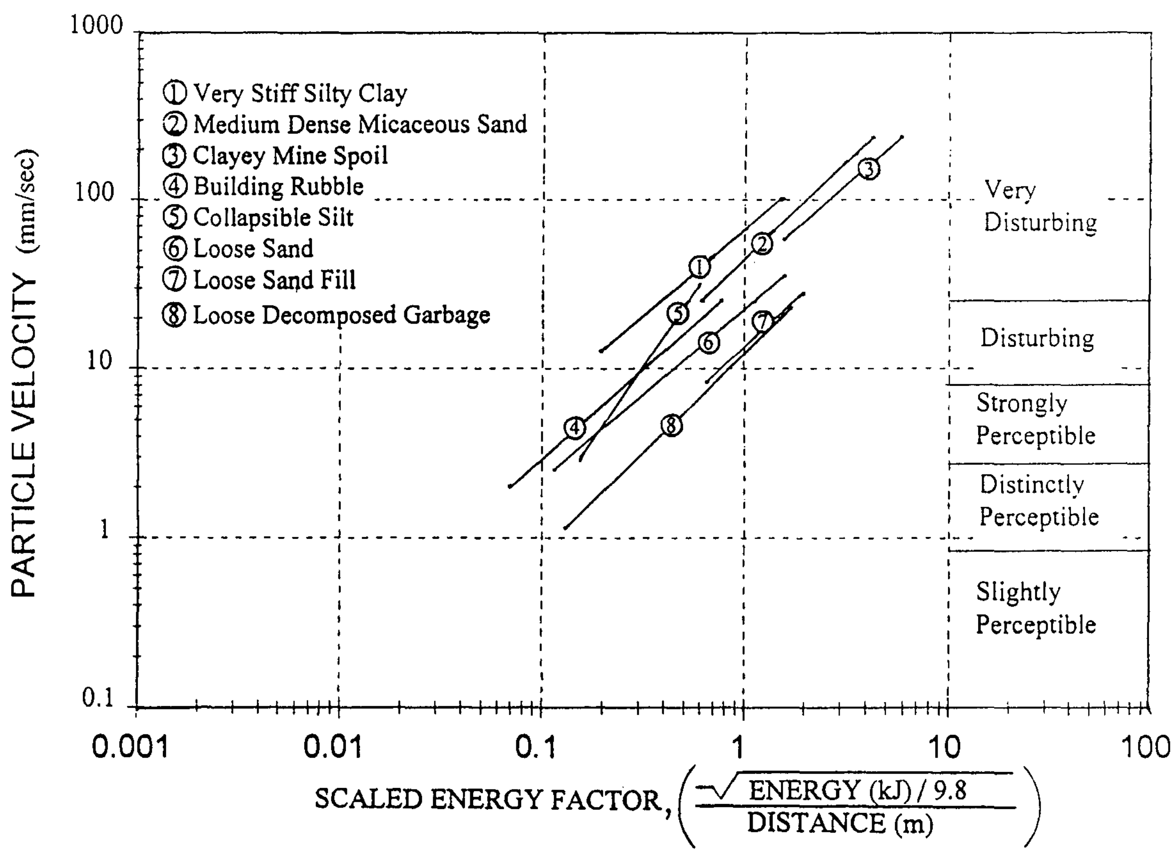

Figure 5. Scaled energy factor versus particle velocity.[7]

If the predicted particle velocity is higher than desired, it will be necessary either to reduce the energy or to increase the distance between the point of impact and the adjacent facility. Either would reduce the scaled energy factor. At some sites, trenches were dug along the property to reduce the particle velocity and this was found to be partially helpful in reducing the surface waves that travel off site. The effectiveness of the trenches can be established at the time of construction from vibration readings taken on the near and far side of the trench following impact of the tamper. Even though damage may not occur, ground vibrations will still be felt by humans; this can be annoying and lead to complaints. The relative response by humans is shown on the right side of figure 5. Ground vibrations would be disturbing to people in the range of 13 to 19 mm/s. Some education on the part of the owner or contractor will be necessary to reduce the fears of the adjacent property owners if the ground vibrations are in this range.

Buried utilities tolerate higher vibration levels than do buildings without damage because the utilities are surrounded in a soil mass. The utility moves with the soil mass and remains confined. Water mains and water pipes have sustained particle velocities of 76 mm/s without damage.

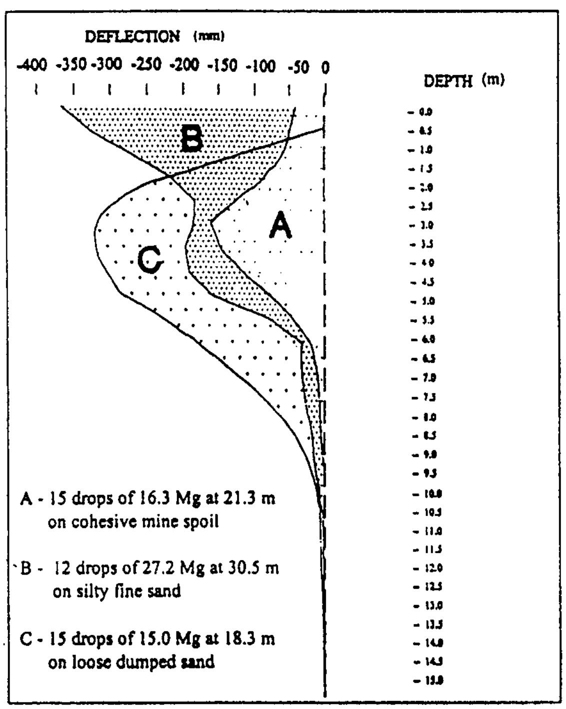

Figure 6.
Lateral movements at 3 m from drop point.

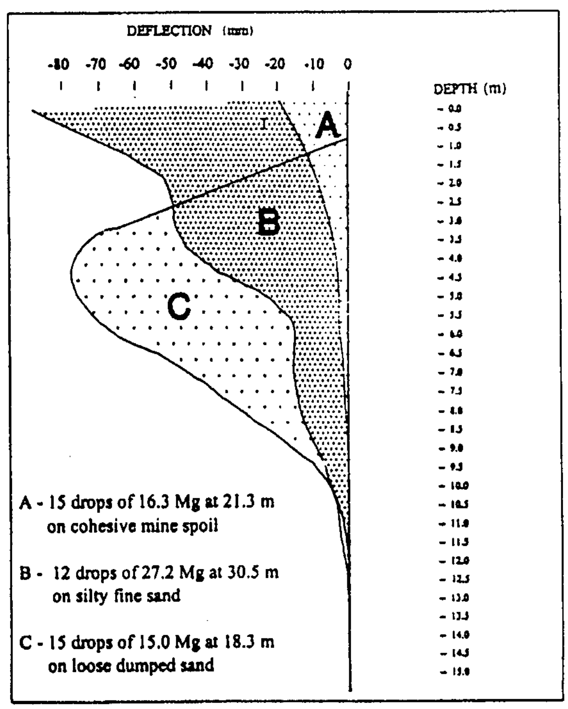

Figure 7.
Lateral movements at 6 m from drop point.

Dynamic compaction also causes lateral ground shifting. Figures 6 and 7 indicate readings taken for three different soil types with inclinometers located at 3.0 and 6.1 m from the point of impact of tampers in the range of 15 to 29 Mg. At a distance of 3.0 m from the point of impact, lateral deformations as much as 254 to 305 mm were observed, while at 6.1 m, lateral deformations range from 13 to 76 mm. These are permanent deformations that occur as the ground is displaced laterally following impact.

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

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

Where dynamic compaction is to be undertaken immediately adjacent to a street or another facility, it may be necessary to erect a shield or barrier to deflect any particles that might otherwise fly off site.

There is a depth limitation on improvement. The heaviest tamper that can be lifted with conventional equipment is about 16 Mg with a drop height of 22.9 to 27.4 m. This will result in a maximum improvement depth of about 11 m. If a greater depth of improvement is required, specialized equipment can be used to lift and drop 27 Mg tampers from a height of 30 m for a predicted improvement depth of about 14 m. If deeper improvement is necessary, dynamic compaction with other systems such as deep grouting and stone columns should be considered.

2.6 ALTERNATE GROUND IMPROVEMENT METHODS

Before deciding upon ground improvement by dynamic compaction, alternate foundation solutions should be investigated in order to compare construction time and costs. Aside from relocation the project to a better location, common foundation alternatives include:

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

- Excavation and replacement of the weak ground. Sometimes this solution may also entail use of sheeting and bracing in confined working areas to retain the adjacent soil mass or dewatering when the water table is high. Contaminated soil deposits could result in a very high excavation cost if the spoil must be taken to a special waste site.

- Preloading of the weak deposits to increase strength and decrease compressibility. This frequently can result in time delays of 6 months to one year to allow the preloading to be effective. Wick drains could be used to accelerate the consolidation process. If materials for surcharging are not available on site, they must be imported from off site and then excavated and removed after the preloading is completed.

- Stone columns could be considered to stiffen clayey soil deposits and vibrocompaction to densify granular soils.

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

CHAPTER 3

CONSTRUCTION AND MATERIALS

3.1 INTRODUCTION

Conventional lifting cranes are used for dynamic compaction projects where the tamper size is less that about 16 Mg. Tampers are sometimes built especially for dynamic compaction while other tampers have been fashioned from steel ingots or from other sources such as a bank vault door.

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

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

3.2 TAMPERS

Dynamic compaction generally is performed with tampers ranging from 5 to 27 Mg. The lighter tampers are used where the thickness of the deposit is relatively thin, such as 3 m, and the heaviest tampers are used where the deposit is about 9 to 12 m thick.

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

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

The contact pressure at the base of the tamper should be 36 to 72 kPa. This pressure is obtained by dividing the weight of the tamper by the area of the base. A significantly higher tamper-pressure, could cause penetration into the ground without densification. Pressures less that about 36 kPa will distribute the energy over too wide an area to cause deep densification.

Figures 8 to 11 are photos of 5.4, 13.6, 16.0, and 27.2 Mg tampers respectively. The 5.4-Mg tamper has a round 152 millimeter thick steel base plate and a steel cylinder that is filled with concrete 25.4 mm thick. The 13.6-Mg tamper also has a large base plate with steel plates stacked on top to making up the remaining portion of the tamper. The plates are welded and bolted together. The 27.2-Mg tamper is solid steel.

Tampers with a low contact pressure are sometimes used to provide surface compaction after the deep densification is finished. The contact pressure at the base of these tampers is typically 10 to 34 kPa. Figure 12 shows a tamper used for the ironing pass. On some projects, surface densification is achieved with either conventional compaction equipment or by proofrolling with a fully loaded dump truck.

3.3 LIFTING EQUIPMENT

To deliver the maximum amount of energy to the ground, the tampers are lifted and dropped using a single cable and a drum on the lifting equipment which has a free spool. The only losses that occur from this type of equipment are friction in the free spool drum, friction of the cable over the upper sheave of the boom, and some air resistance at the base of the tampers. Figures 13 to 15 show typical equipment used to raise and drop the tamper. There are also specialized pieces of equipment that will raise the tamper with multiple-part lines. Then a release mechanism, allows the tamper to free fall. This also is an acceptable method for applying the energy.

Conventional lifting cranes generally are used for tampers in the size range of 5 to 16 Mg. Table 1 shows the type of crane and cable size required to repeatedly raise and drop tampers of different size. For tamper weights in the range of 16 to 23 Mg, the conventional lifting equipment must be modified to prevent breakdowns. When the tamper is released, a rocking motion causes a stress in the shaft connecting the cab to the tracks. To reduce this stress, spuds are frequently placed on the rigs to reduce the rocking. In addition, the lifting drums are enlarged and made thicker to withstand the forces.

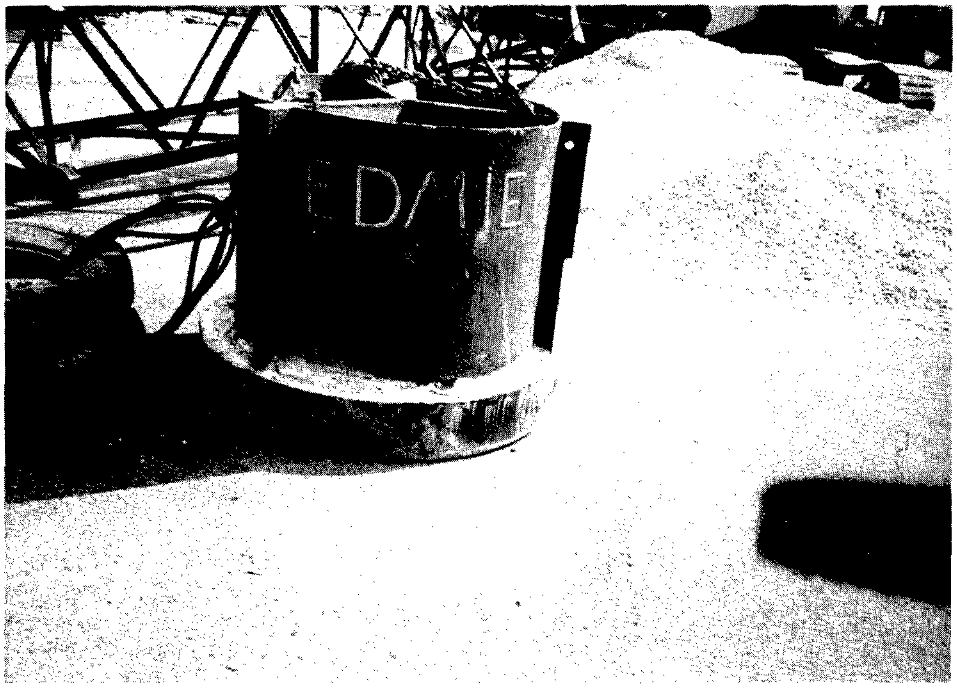

Figure 8. 5.5 Mg tamper.

Figure 9. 13.6 Mg tamper.

Figure 10. 14.0 Mg tamper.

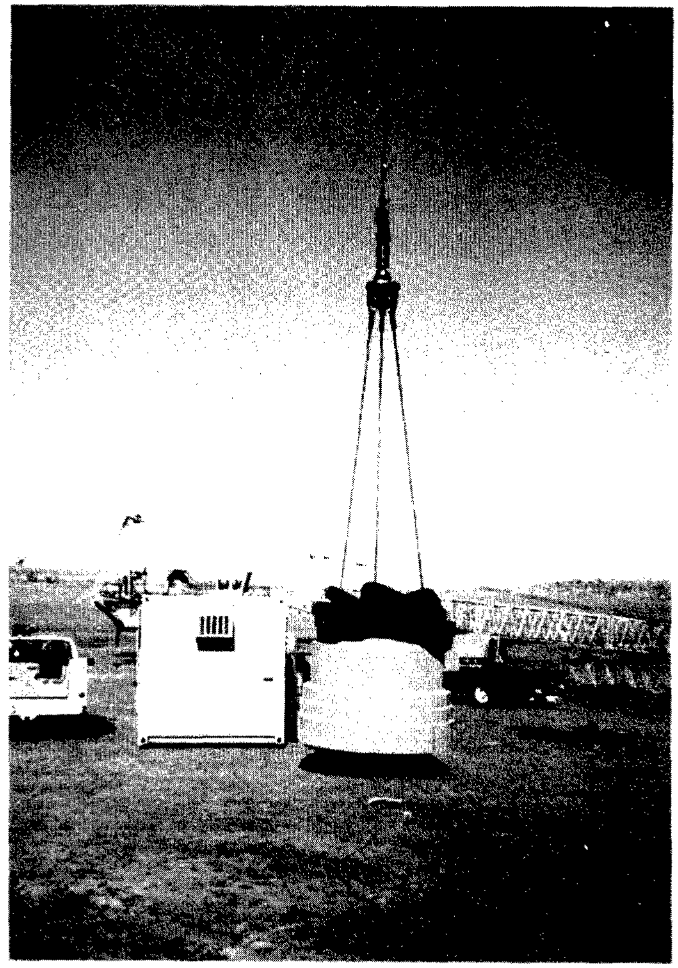

Figure 11. 32.0 Mg tamper.

Figure 12. Tamper with low contact pressure for ironing pass.

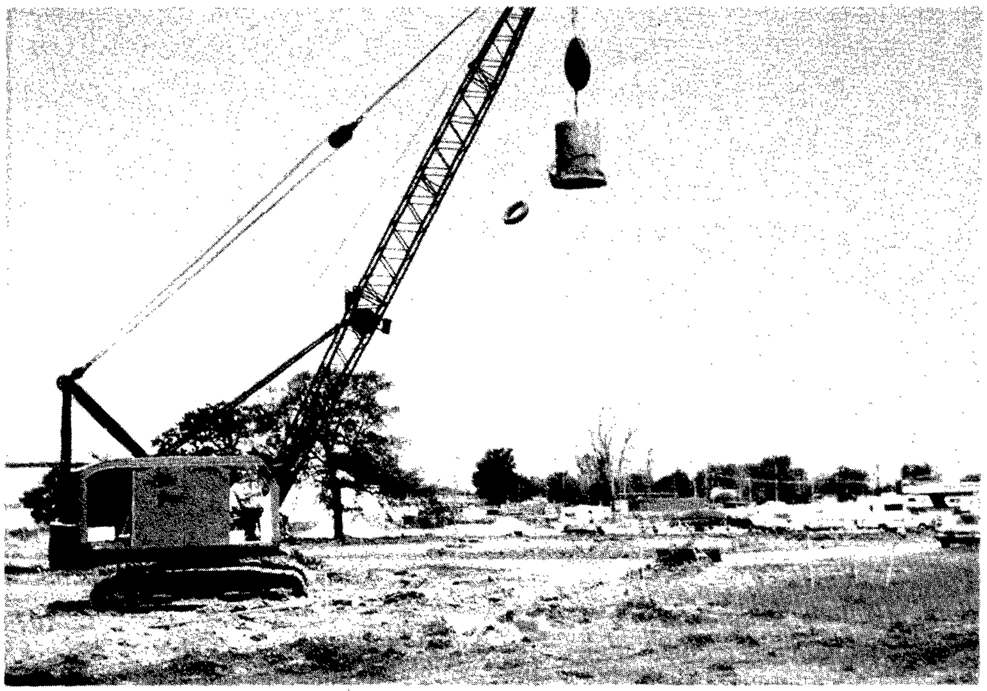

Figure 13. Lifting crane of 36 Mg rated capacity used with 5.5 Mg tamper.

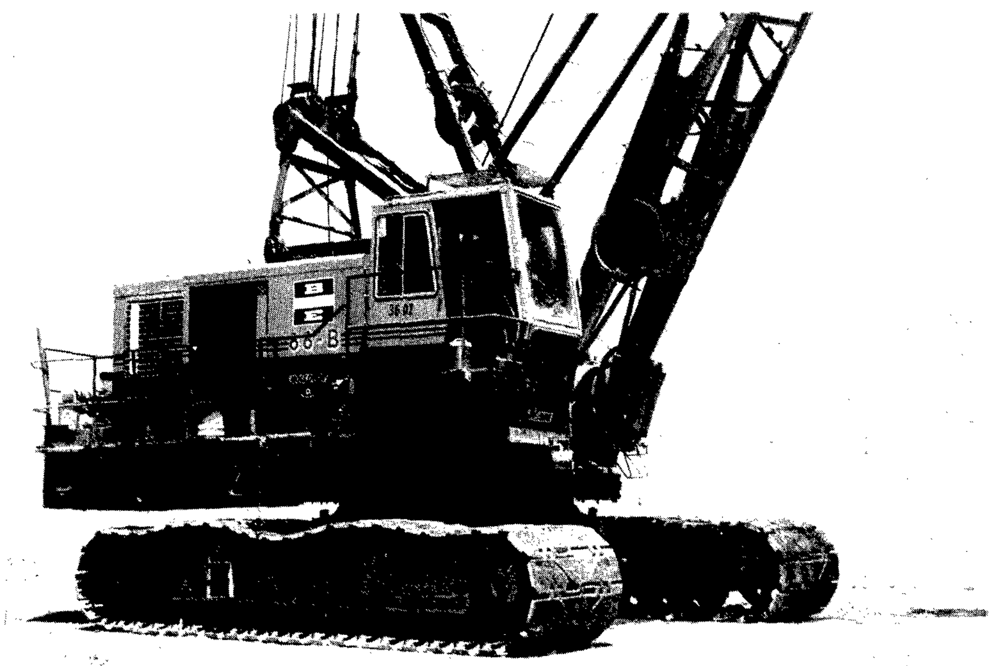

Figure 14. Dragline type lifting crane with 32 mm cable for use with 18.2 Mg tamper.

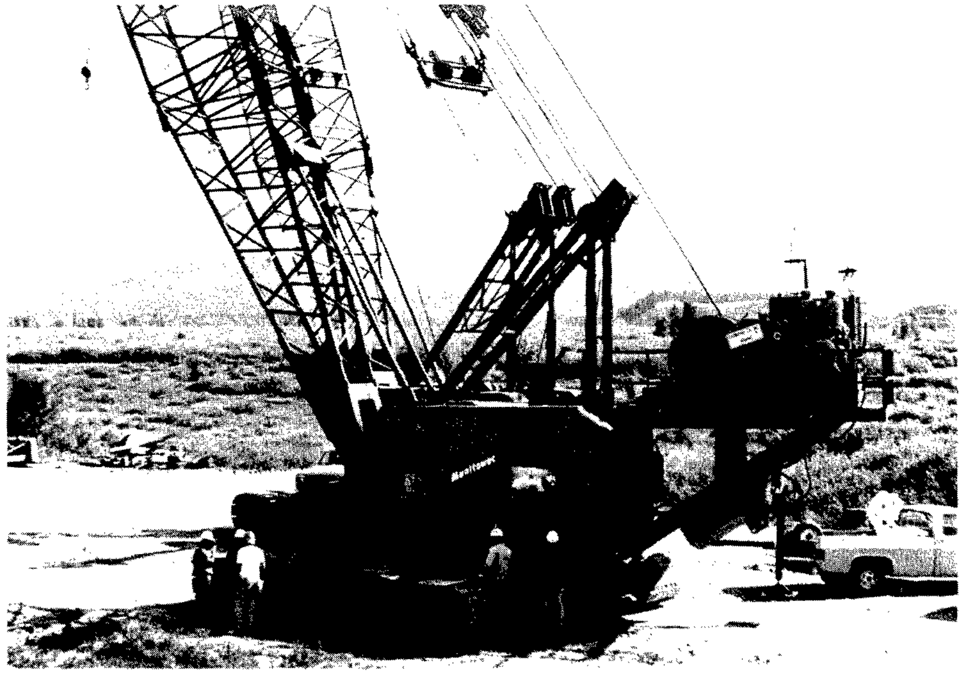

Figure 15. Manitowoc 4000 crane with special lifting drum attached to rear of crane for use with 29 Mg tamper.

Table 1. Equipment requirements for different size tampers.

Tamper Weight	Crawler Crane Size	Cable Size (mm)
3.5 to 7.5 Mg	40 to 50 tons	19 to 22
7.5 to 13 Mg	50 to 100 tons	22 to 25
13.5 to 16.5 Mg	100 to 125 tons	25 to 29
16.5 to 23 Mg	150 to 175 tons	32 to 38

NOTE: 1 ton = 0.91 metric tonnes (t)

Specialized lifting equipment has been devised for tampers 23 to 27 Mg. Figure 16 shows a crane developed for large-size tampers. The lifting drum is approximately 1.5 m in diameter. The upper carriage of the crane is fixed to the tracks so the crane cannot rotate in a horizontal direction. This reduces damage to the connection between the cab and the tracks but makes maneuvering around the site more difficult.

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

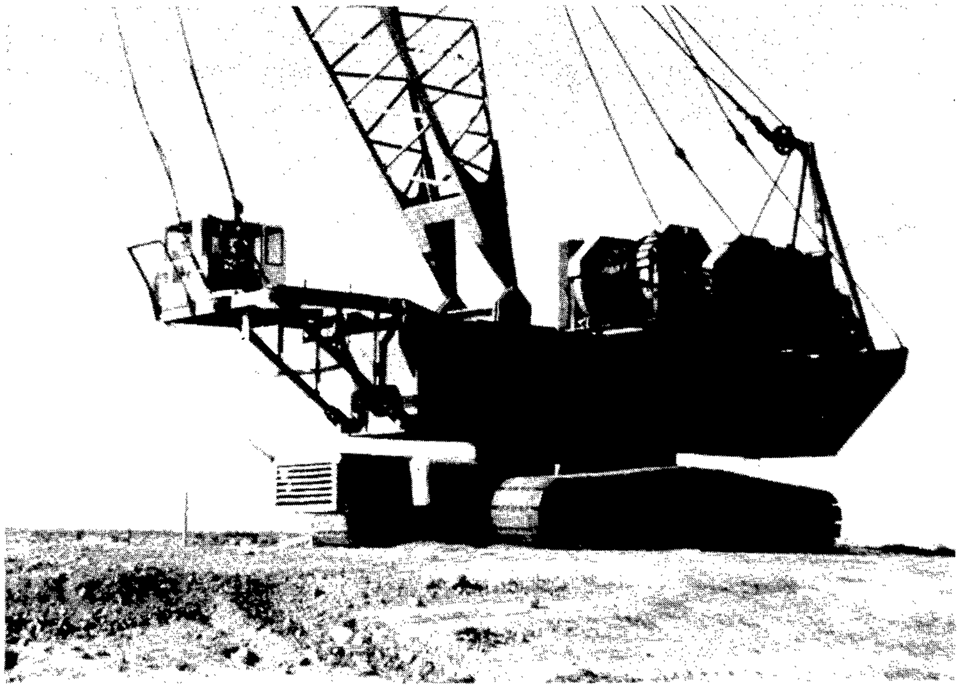

Figure 16. Lifting crane built only for dynamic compaction with 1.5 m diameter lifting hoist and single line-rated capacity of 45 Mg at line speeds of 26 m/min used with 29 Mg tampers.

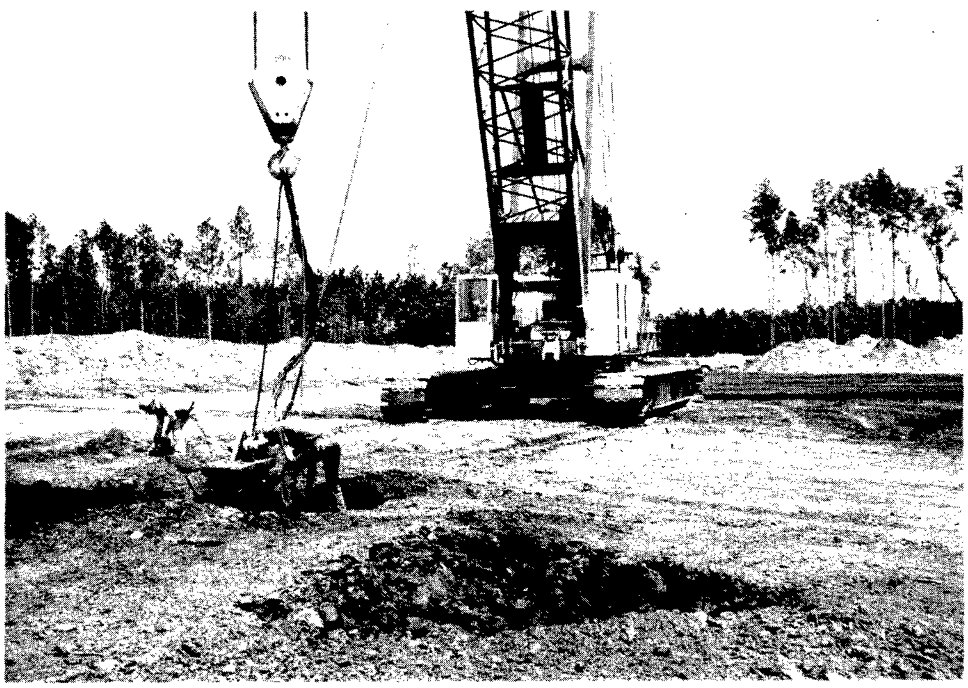

Figure 17. Multiple part line used to lift 29 Mg tamper (below grade in photo).

3.4 SURFACE STABILIZING LAYER

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

The stabilizing layer usually consists of a granular material with a typical gradation range from 152 mm maximum size, down to sand size. The thickness of these layers depends upon the stability of the surface deposits, but thicknesses ranging from 0.3 to 0.9 m have been used successfully. Figure 18 shows a dynamic compaction equipment working on a site where crushed rock was placed to a depth of 0.9 m over a landfill.

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

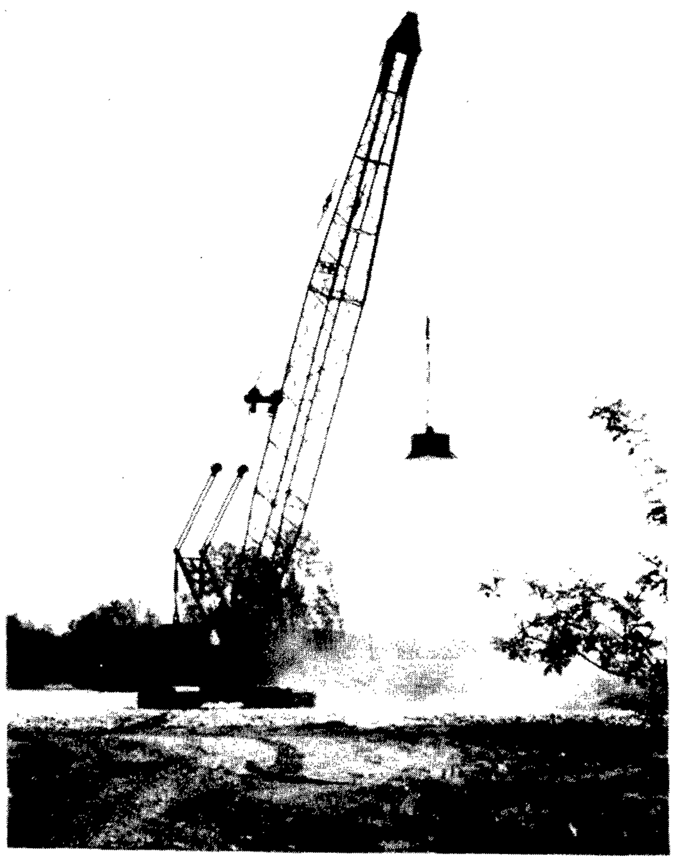

Figure 18. Dynamic compaction over 0.7 m layer of crushed rock placed on landfill.

3.5 CONSTRUCTION SEQUENCING

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

A phase is the application of the energy in a specific pattern. For instance, initial energy applications undertaken on 7.6 m centers across the area, could be labelled phase 1. Phase 2 could be the application of energy midway between initial grid points. A single phase of energy application is most often used on projects where pore pressures dissipate rapidly and the energy can be applied on grid point locations immediately adjacent to a grid point that has just been densified. Multiple phases are used on projects where pore pressures rise and take some time to dissipate; therefore it is preferable to apply energy on an intermittent grid pattern.

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

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

CHAPTER 4

DESIGN CONCEPTS

4.1 PROBLEM DEFINITION

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

- Determining the project performance requirements for the completed structure.
- Selecting the tamper mass and drop height to correspond to the required depth of improvement.
- Estimating the degree of improvement that will result from dynamic compaction.
- Determining the applied energy to be used over the project site to produce the improvement.

The information presented in this chapter is a brief summary of the design concepts. FHWA-SA-95-067 provides additional details.

4.2 PERFORMANCE REQUIREMENTS

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

As an example, if a roadway embankment is to be constructed over weak ground, one concern is settlement of the embankment. Most embankments can withstand up to 150 mm of settlement without detrimental performance of the pavement system, provided the settlement is reasonably uniform, doesn't occur next to a pile-supported structure, and occurs very slowly.[10] Based upon the estimated properties of the soil following densification, a settlement prediction can be made for the height of embankment to be constructed, to determine if the settlement will be less than 150 mm. If so, dynamic compaction would satisfy the project requirements. Conversely, if the predicted settlement using conventional analysis would exceed what the embankment can tolerate, other methods of supporting the embankment should be considered.

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

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

The procedures in this chapter for estimating improvement are conducted before the project begins, but verification of the improvement by additional field is required. After dynamic compaction is completed, borings are generally made to determine the important design properties.

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

4.3 DEPTH AND DEGREE OF IMPROVEMENT

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

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

Where: D=depth of improvement in meters

n=empirical coefficient that is approximately 0.5 for most soils. As shown in figure 19, n varies between 0.3 and 0.8.

W=mass of tamper in Megagrams

H=drop height in meters

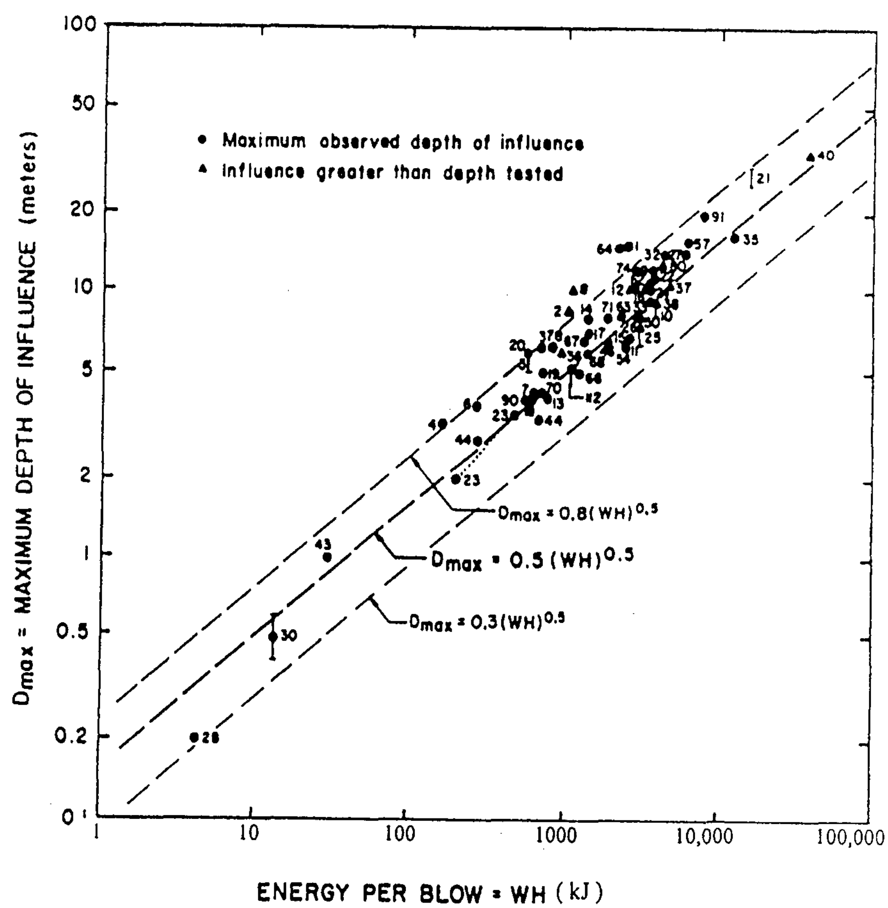

Figure 19. Trend between apparent maximum depth of influence and energy per drop.

Other factors affecting the depth of improvement include the presence of soft energy absorbing layers. Hard layers at the surface that do not allow the energy to be transmitted to greater depths as improper application of the energy. A more thorough discussion of factors influencing the depth of improvement are presented in references 7.

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

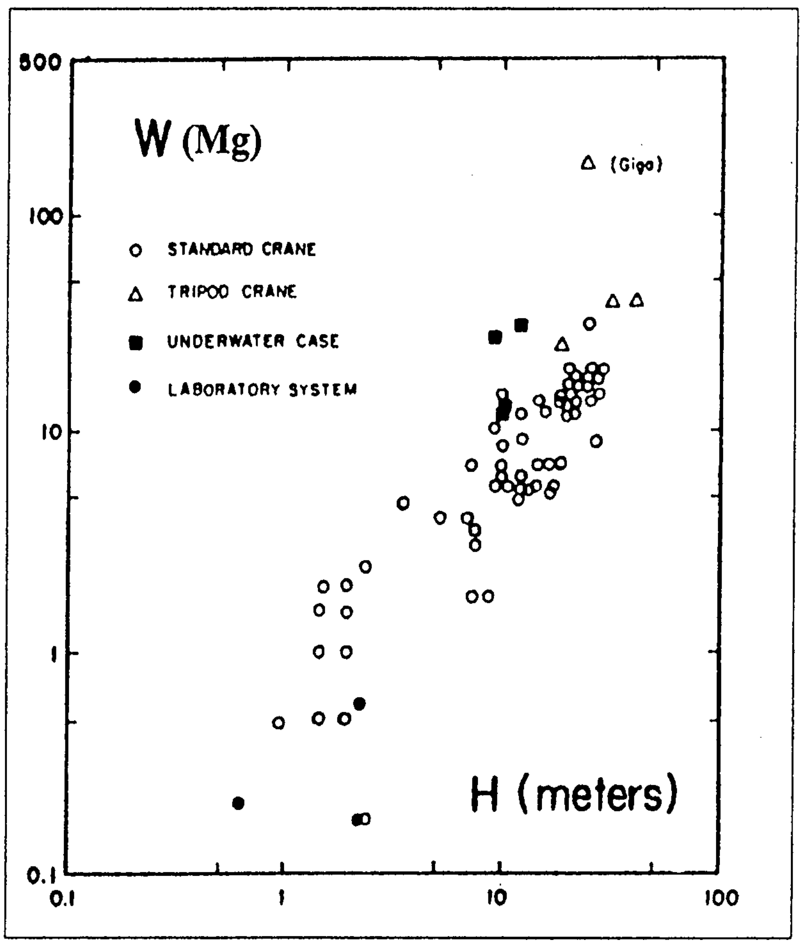

Figure 20. Relationship between tamper mass and drop height.

The amount of improvement resulting from dynamic compaction is generally measured by conventional in-situ testing techniques such as SPT, CPT, or PMT. Test values obtained after dynamic compaction are compared with initial values before dynamic compaction, to monitor the improvement. The greatest amount of improvement is generally near the upper portion of the soil layer densified and then decreasing with depth. Figure 21 illustrates the variation of improvement with depth.

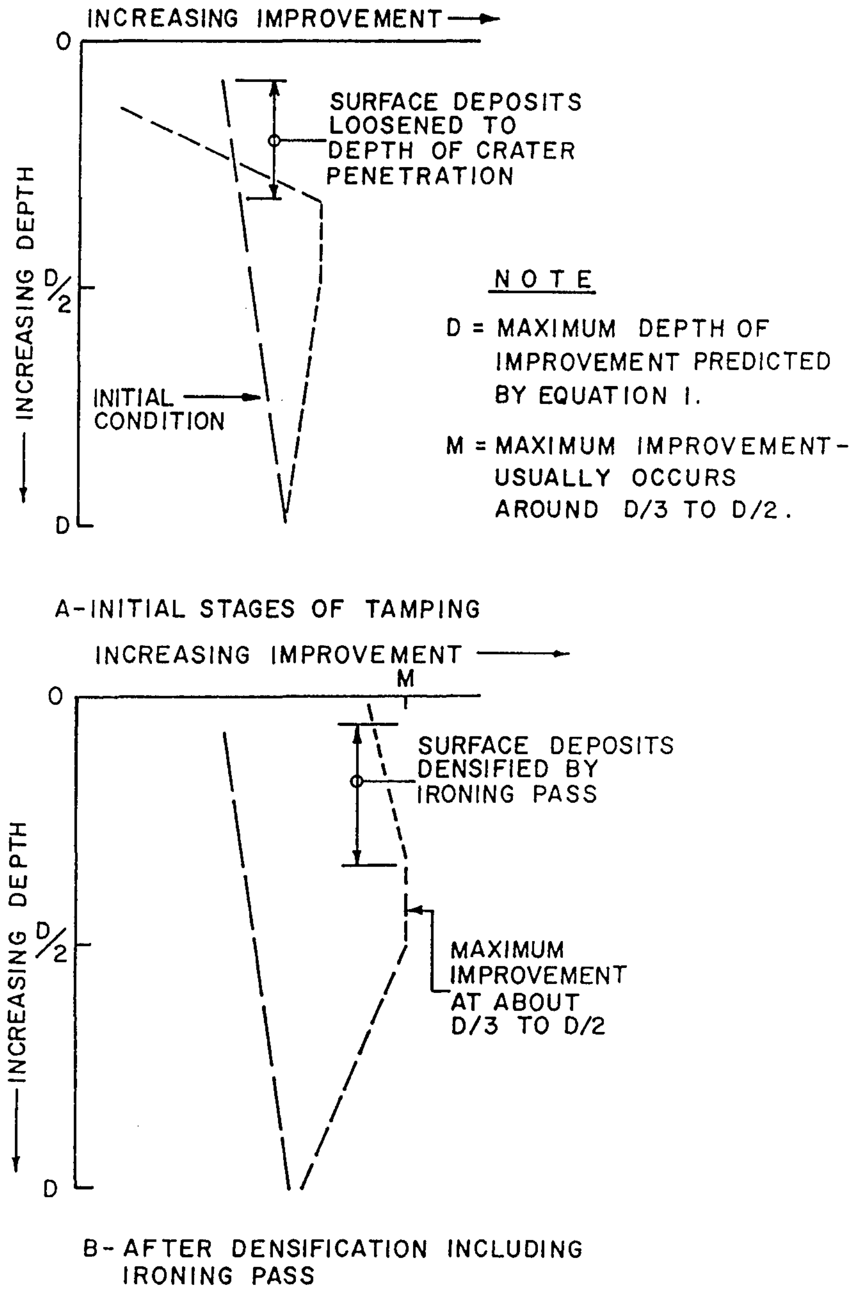

Figure 21. Variations in improvements with depth during dynamic compaction.

The degree of improvement achieved is primarily a function of the average energy applied at the ground surface. In general, the greater the amount of energy, the greater the degree of improvement. However, there are limitations to the maximum SPT, CPT or PMT values that can be reached.

Based upon a review of a number of project sites, upper bound test values of SPT, CPT, and PMT tests are shown in table 2.[7] These indicate maximum values of improvement at depths of D/3 to D/2. Above and below this depth range, the test values would be less. At project sites where these test values were measured, the average energy applied was typically on the order of 2 to 3 MJ/m^2. If a lesser amount of energy is applied or if there is some other complicating factor such as energy absorbing layers or a hard surface crust that does not allow the energy to penetrate deeper, the degree of improvement could be significantly less than shown in table 2.

Table 2. Upper bound test values after dynamic compaction.

	Maximum Test Value		
Soil Type	**Standard Penetration Resistance (blows/300 mm)**	**Static Cone Tip Resistance (MPa)**	**Pressuremeter Limit Pressure (MPa)**
Previous coarse-grained soil:			
sands & gravel	40 - 50	19 - 29	1.9 - 2.4
Semipervious soil:			
sandy silts	35 - 45	13 - 17	1.4 - 1.9
silts & clayey silts	25 - 35	10 - 13	1.0 - 1.4
Partially saturated impervious deposits:			
clay fill & mine spoil	30 - 40*	N/A	1.4 - 1.9
Landfills	20 - 40*	N/A	0.5 - 1.0

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

The upper bound test values given in table 2 should be used only for estimating the maximum degree of improvement that is possible at a project site. The actual improvement should be measured by field tests such as SPT, CPT, or PMT tests after dynamic compaction. Some testing should be conducted while the dynamic compaction equipment is at the site so additional energy application can be applied if the degree of improvement has not reached the desired value. However, it has been found that there is a delayed improvement with time following dynamic compaction after all excess pore water pressures have dissipated. This strength gain with time has been attributed to cementation between soil grains, to secondary consolidation, and to aging. This phenomenon occurs mostly in fine-grain soils but has also been measured in granular soils. Thus, if the tests taken at the end of dynamic compaction are only borderline acceptable, additional testing could be conducted at a later date to confirm the improvement in soil properties with time.

A further discussion of the estimated improvements from dynamic compaction and strength gain with time is discussed in FHWA-SA-95-067.

4.4 ENERGY REQUIREMENTS

Dynamic compaction is generally undertaken on a grid pattern throughout the area. For this reason, it is convenient to express the applied energy in terms of average values. This average applied energy can be calculated on the basis of the following formula:

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

Where: AE=average applied energy in J/m^2

N=number of drops at each specific drop point location

W=tamper mass in Mg

H=drop height in m

P=number of passes

If the different sized tampers and drop heights are used, the average applied energy would be the sum of all the levels of effort. For example, the high level of energy is generally applied with a heavy tamper and a high drop height. This is frequently followed with a low level of energy (called an ironing pass) using a smaller size tamper and a lower drop height. The average applied energy would be the sum of the energy imparted to the site.

Most projects have been completed using energy levels ranging from approximately 1 to 3 MJ/m^2. However, the energy can be varied for each project site depending upon the degree of improvement needed to satisfy the project requirements.

During the initial design or planning stages, an estimate of the required energy can be obtained using table 3. This table incorporates some of the variables that influence the amount of energy required to achieve adequate improvement. This includes the type of soil being densified, the initial relative density of the deposit, and the thickness of the deposit.

Table 3. Applied energy guidelines.

Type of Deposit	Unit Applied Energy (kJ/m^3)	Percent Standard Proctor Energy
Pervious coarse-grained soil (zone 1 of figure 5)	200 - 250	33 - 41
Semipervious fine-grained soils (zone 2 and clay) fills above the water table (zone 3 of figure 5)	250 - 350	41 - 60
Landfills	600 - 1100	100 - 180
Note: Standard Proctor energy equals 600 kJ/m^3		

The deposits are grouped into three broad categories of soils with landfills requiring the greatest amount of energy and coarse grained soils the least. The thickness of the deposit has been considered in table 3 by normalizing the applied energy by the thickness of the deposit. The initial relative density of the deposit is taken into account by showing a range in the amount of energy required. Within any particular soil type, the loosest deposits would require the higher level of suggested energy and the denser deposits the lower amount of suggested energy.

An example helps to illustrate the use of table 3 for planning purposes. Consider a site that consists of a loose rubble fill with a thickness of 8 m. The rubble fill can be classified as a pervious coarse grain soil typical of zone 1. Because the deposit is in a loose condition, the upper bound of applied energy for this deposit of (250 kJ/m^3) would be used. Since the deposit is 8 m thick, 2 MJ/m^2 of average applied energy is required. Substituting this number into equation 2, and using the tamper mass and drop height from equation 1, different combinations of drops, grid spacing, and passes can be evaluated.

To determine the product of the number of drops, N, and the number of passes P, to obtain the calculated applied energy use equation 2. The required mass of tamper, W, and drop height, H, is calculated from equation 1. The grid spacing usually is selected as 1.5 to 2.5 times the diameter of the tamper. Equation 2 is solved for N x P. It is more efficient for the contractor to apply all the energy in on pass than multiple passes. Using P=1, the number of drops at each grid point can be determined.

Multiple passes are used when not all the drops can be made at each grid point at one time, while the product of N and P remains the same whether there is a single pass or multiple passes.

The energy requirements shown in table 3 can be related to standard proctor energy, which is equal to 600 kJ/m^3. Pervious coarse grain soils generally require about 33 to 41 percent of this energy to achieve adequate densification, and this is caused by some densification that occurs naturally in these deposits after they have been in place for a while. Conversely, landfill deposits require 100 to 180 percent of the standard proctor energy. Landfills are generally in an extremely loose condition and may even be underconsolidated so a significantly higher level of energy is required to densify these formations.

Close monitoring of the field operations coupled with SPT, CPT or PMT tests during dynamic compaction should be used to verify if the desired improvement is reached.

CHAPTER 5

CONTRACTING PROCEDURES, SPECIFICATIONS, AND CONSTRUCTION MONITORING

This chapter presents a brief summary of methods that can be used in contracting for dynamic compaction plus an overview of the monitoring that normally occurs during the field work. Complete construction specifications are detailed in FHWA-SA-95-067.

5.1 CONTRACTING PROCEDURES

The designer will develop a set of drawings and specifications for the contracting package. This information is used by the construction personnel and establishes the basis of payment and details of the quality assurance program. Two contracting approaches are available in preparing the plans and specifications:

- A method of approach.
- A performance approach.

With the method approach, the designer outlines the work plan that the contractor is to follow. Information is provided on the dynamic compaction process, including the size of tamper, the drop height, the average energy to be applied, whether the energy must be applied in single or multiple passes or phases, the need for a working mat, and all other facets of dynamic compaction that are necessary to achieve the proper improvement.

The method specification approach is more common as knowledge of dynamic compaction and the amount of energy to apply becomes better known among designers. The method specification also allows for general contractors to undertake some of the work.

With the performance approach, the contractor is responsible for achieving the desired result. The designer in this case specifies the required depth and degree of improvement and also provides information regarding the site and soil conditions. The contractor is free to select the size of the tamper, the drop height, and other specifics of dynamic compaction to achieve the requirements set forth by the designer.

Only experienced contractors should be allowed to bid work under a performance approach. There are approximately five to eight specialty contractors in the United States who specialize in dynamic compaction. Since these contractors will do a significant amount of engineering and planning of the dynamic compaction operation, the cost for dynamic compaction using this approach is generally higher than for projects where method specifications are used.

5.2 METHOD SPECIFICATION

When preparing a method specification, the designer should include the following items in the plans and specifications:

- Tamper mass and size.
- Drop height.
- Grid spacing.
- Applied energy.
- Number of phases or passes.
- Site preparation requirements.
- Surface compaction after dynamic compaction.
- Drawings of the working area.
- Subsurface investigation data.

The owner or designer is responsible for:

- Monitoring during construction.
- Borings and tests after dynamic compaction.

The contractor is responsible for:

- Providing adequate equipment to complete the work in a timely manner.
- Safety of personnel and equipment.
- Work plan subject to approval by designer.

5.3 PERFORMANCE SPECIFICATION

If a performance specification is prepared, the designer should specify the desired end product including:

- Minimum soil property value to be achieved and the method of verification.
- Maximum permissible settlement.
- Other objectives of site improvement.
- Minimum contract qualification requirements.

The owner or designer also provides the initial subsurface data and the lateral extent of the project site. Some site monitoring is also required as it is in the interest of the owner to be aware of the details of the field operations, including any changes in the planned scope.

The contractor is required to meet the minimum specified final product and is responsible for:

- Proper equipment and a work plan.
- Meeting the project deadline.
- Safety.
- Field monitoring.
- Additional subsurface exploration as required to properly prepare a dynamic compaction plan.
- Verification of the end product.

5.4 QUALITY CONTROL MONITORING

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

During dynamic compaction, close observation of the procedures should be undertaken because it is frequently necessary to adjust the field program. Reasonable and practical adjustments can only be made if good monitoring information is available. Monitoring during dynamic compaction includes:

- Measuring crater depths and adjacent ground surface heave at occasional drop point locations to determine the proper number of drops that can be applied at a location for maximum efficiency. As an example, the first few drops generally result in significant crater displacements without surrounding ground heave. After six or seven drops at one point, the crater depths may become significantly less for each drop, and some ground heave may occur. At some point, the additional crater volume produced by a single drop may be equaled by ground heave adjacent to the drop point location, which means that there is no longer any compaction occurring in the soil mass. Because there is merely a ground displacement at this time, application of additional energy would not produce densification.

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

- Field testing could be performed while the dynamic compaction is under way to confirm that the desired improvements are being reached. This could include soil borings with SPT, CPT, or PMT test. At some sites, such as landfills, in-situ soil testing is often meaningless and load tests are frequently made with settlement readings generally taken over a period of at least 1 week to measure performance.

  Field testing is an indicator of the strength and compressibility of the deposits shortly after dynamic compaction. The properties of some soils improve with time, which must be kept in mind when evaluating the result.

- Where there are fine grained materials such as silts or clayey silts, it may be helpful to install piezometers in the deposits below the water table. The purpose of this monitoring is to determine the magnitude and rate of dissipation in excess pore water pressure during dynamic compaction. This would determine the waiting period between drops applied at a location.

- Where dynamic compaction is planned adjacent to structures, vibration readings should be taken to determine if ground vibrations pose a potential risk for the buildings. Building damage has been correlated to peak particle velocities as measured by seismograph and there are charts available for determining permissible levels of vibrations.

- In addition to the testing described above, qualified personnel should make general observations of the dynamic compaction operations. Adjustments in the field tamping program can be made depending upon these observations. For instance, where ground subsidence much greater in one area than in the remaining areas, this could indicate an extremely loose pocket that requires additional tamping. Excessive ground heave in other areas might indicate soils that will not properly compact, which may require either a greater waiting period between densification or partial undercutting and replacement with a soil that will densify under dynamic compaction.

After the dynamic compaction is completed, conduct additional field explorations to confirm the degree and depth of improvement. This investigation is undertaken with soil borings along with SPT, CPT, or PMT tests. Because improvements in SPT, CPT, and PMT values have been observed 30 to 60 days following completion of dynamic compactions, it would be helpful if borings and field testing could be undertaken at that time.

On some projects, settlement plates and inclinometers have been installed following dynamic compaction to monitor the movement of the subsoils during construction of the embankment or structure. While not necessary, this practice does provide useful information to evaluate the effectiveness of the dynamic compaction.

For projects completed under a method type of specification, the owner and designer are responsible for monitoring operations. This would include providing field personnel during the dynamic compaction to confirm the depth and degree of improvement.

For projects completed under a performance type specification, the contractor is responsible for providing field monitoring and providing borings with SPT, CPT, or PMT tests after dynamic compaction to confirm the depth and degree of improvement. However, it is in the owner's interest to have field personnel on the site because any changes to be made in the dynamic compaction plan would need to be mutually agreed upon between the owner and the contractor. The borings and field tests made after dynamic compaction should also be monitored to confirm that the improvement has been achieved.

CHAPTER 6

CASE HISTORIES

Two case histories are presented to illustrate typical applications.

6.1 HIGHWAY EMBANKMENT CONSTRUCTED IN MINE SPOIL

Project Description

Interstate 65 in Jefferson County, Alabama was extended over a mine spoil area for approximately 762 m. At either end of the mine spoil area, the final grades required approximately 4.6 m of new fill, but in the center, there was a cut of 13.7 m. From this final grade, there would still be 15.2 to 36.6 m of mine spoil below the roadway. The new roadway was to be a four-lane highway with a width of 55.5 m.

Soil Conditions

The mine spoil is classified as a mixture of rock fragments of shale, siltstone, and sandstone embedded within the soil matrix of silt and sand or clayey silt and sand. Approximately 50 percent of the mine spoil was larger than 51 mm.

The area was strip mined from 1977 to 1980. The initial subsurface investigations for the interstate occurred in 1983, and construction of the roadway started in 1984. Thus, the mine spoil was of relatively recent age at the time of construction.

Soil borings were made through the mine spoil using standard penetration (SPT) tests. Excluding the very high SPT values where weathered chucks of rock were encountered, the majority of the mine spoil had SPT values ranging from 15 to 20 blows per 0.3 m. There were some areas where the SPT values were as low as 5 to 10 blows per 0.3 m and other areas as high as 25 to 35 blows per 0.3 m. The variation in SPT value at depth is illustrated in figure 22.

Ground water was encountered at depths in excess of 30.5 m below ground surface and was not a factor on this project.

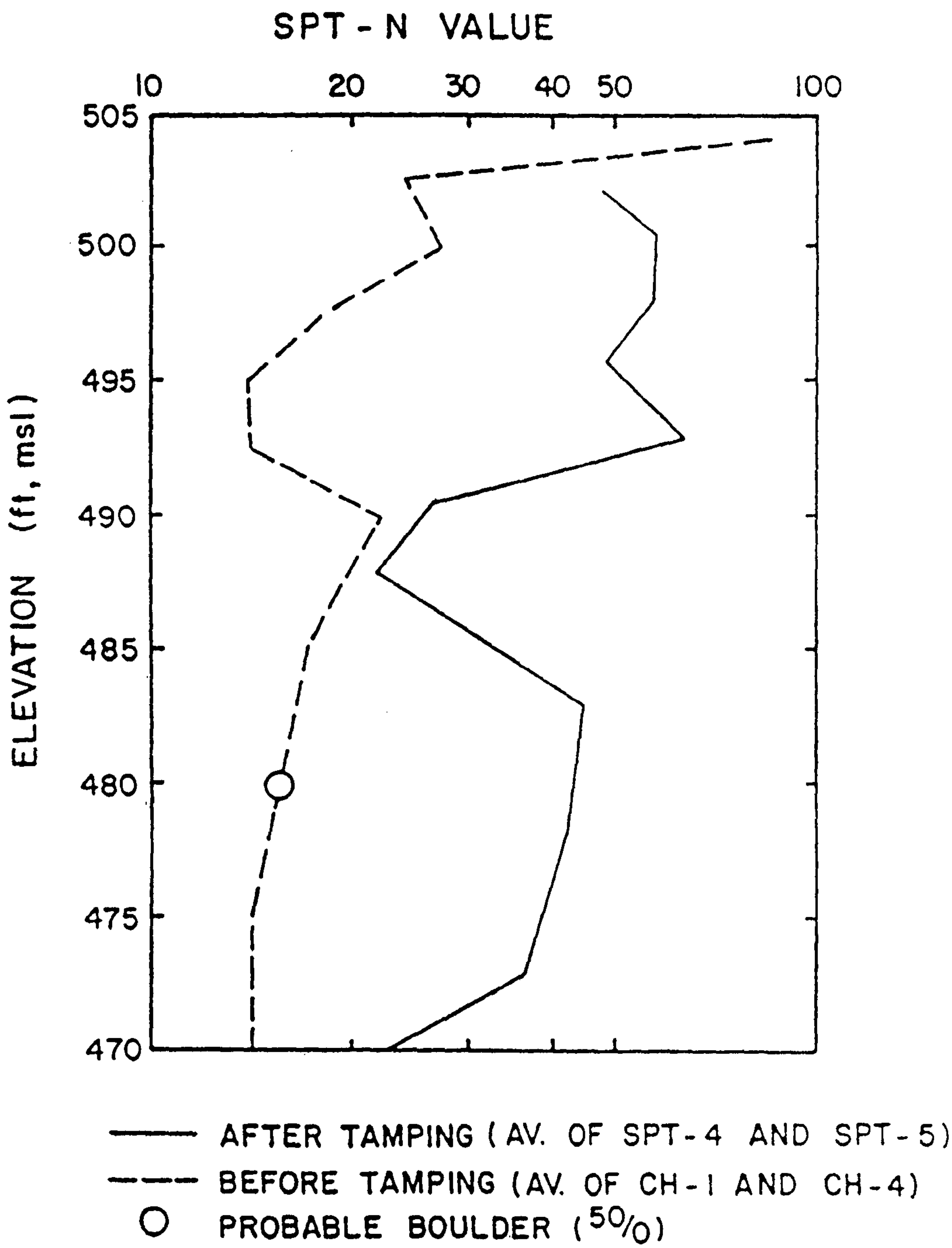

Figure 22. Increase in SPT values in a mine spoil after dynamic compaction.

Design Concerns

The Alabama Highway Department and its consultants realized that there were large variations in material classifications, strengths, and compressibilities in relatively short vertical and horizontal distances over the entire site. For this reason, site improvement by dynamic compaction was recommended in order to:

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

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

3. Densify localized areas of loose/soft soil in the upper 7.6 to 9.1 m, which could subside after highway construction.

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

Predicted Densification Procedure

Using the guidelines presented in this technical summary, the first step is to calculate the tamper mass and drop height for a desired depth of improvement of 7.6 to 9.1 m.

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

$$9\ \text{m} = 0.5(WH)^{1/2}$$
$$WH = 324$$

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

The second step is to calculate the energy to apply. Using table 3 as a guide, for zone 2 type deposits,

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

Because the deposits are already in a medium-dense condition, use E=250 kJ/m^3. For a 9-m-thick deposit, applied energy = 9 m *x* 250 kJ/m^3 = 2250 kJ/m^2.

The third step is to determine the grid spacing and number of drops. Assuming a 20 Mg tamper dropped 16.5 m, a reasonable spacing would be 3 m as the tamper diameter is 2 m. Using equation 2, and assuming only one pass and an ironing pass which will apply 250 kJ/m^2 energy, the number of drops can be computed.

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

For the grid spacing of 3 m *x* 3 m, N=5.5; or six drops at each grid point.

After dynamic compaction, the maximum SPT value according to table 2 would be 34 to 45 for sandy silts and 25 to 35 for clayey silts. Estimated induced settlement is 5 percent (9 m) = 0.45 m.

Actual Densification Procedure

Based upon results of a test section, it was determined that densification to a depth of 7.6 to 9.1 m could be achieved using a 20-Mg tamper with a drop height of 19 m. The high-level energy was applied using five drops at each grid point location, with a spacing of 3.0 m between grid points.

After the high level energy was applied, the ground surface was levelled and an ironing pass using the same tamper with a drop height of 5.8 m, and a grid spacing of 1.8 m, and one drop per grid point location.

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

Ground Improvement

In figure 22, soil borings with SPT values made after dynamic compaction are compared to the borings with SPT values before dynamic compaction. This data indicates the improvements were obtained to depths of approximately 10.7 m and that the SPT values increased significantly.

Another indication of ground improvement was the amount of induced ground settlement by dynamic compaction. Within the test sections, ground elevations were taken on a grid patter and measured following various levels of energy application. The data are summarized on figure 23.

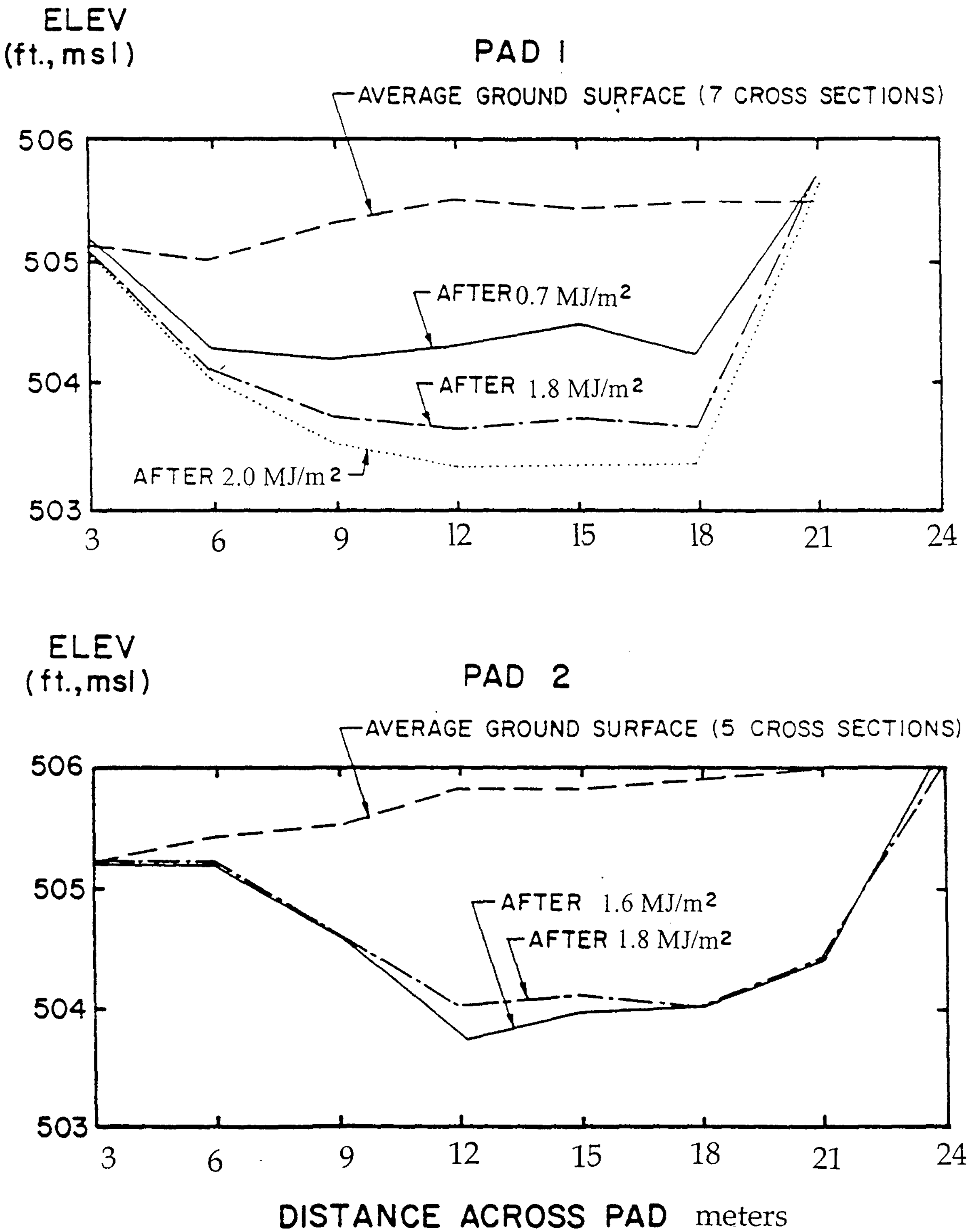

Figure 23. Induced settlement following dynamic compaction at the mine spoil project in Alabama.

At test pad 1, the average ground subsidence following full application of the primary energy was approximately 0.6 m. At test pad 2, approximately 0.6 m of induced settlement was observed after energy application corresponding to about 90 percent of the prescribed energy.

During the production phase of dynamic compaction, the average induced ground settlement was 0.5 m, although it could have been more because some fill was brought in during the leveling of the craters. In local areas, the average induced settlement was significantly higher. Figure 24 illustrates the variation in the crater depths that were observed in certain sections of the dynamically compacted area. The normal average crater depth for the mine spoil was 1 to 1.1 m, but crater depth as high as 1.5 to 2.7 m occurred in some locations, indicating a soft or void area. Additional high-energy tamping was undertaken in the soft area after ground leveling and placement of fill to raise the grade.

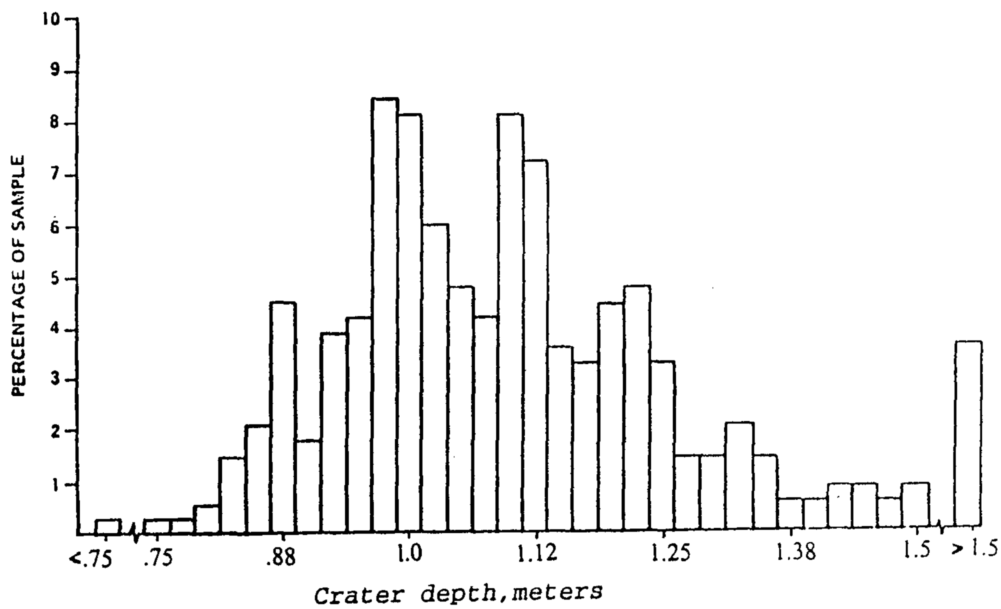

Figure 24. Statistical variation in crater depths at the mine spoil site in Alabama.

Contracting Procedure and Cost

A method specification was prepared for this project, and non-specialty as well as specialty contractors were allowed to bid. The project was awarded to an excavating contractor. After an initial 2-week trial period with some experimentation, the work proceeded on a reasonably good schedule. One hundred working days were required to dynamically compact approximately

37,200 m^2 for an average of 372 m^2 per day. When considering only production time, the average tamping rate was 63 impacts per hour for the primary phase and 67 impacts per hour for the ironing pass.

The project was bid on a price per drop that included an overall mobilization charge for all facets of the embankment construction. Therefore, the portion of the mobilization attributed to dynamic compaction is difficult to determine. The bid price per drop was $2.90 for the high energy phase and $2.65 per drop for the ironing pass. Using the prices given and an estimate for the mobilization, the cost for the dynamic compaction was approximately $7.20 per m^2 of area treated. This amounted to about 4 percent of the total project cost.

The estimated cost for excavation of the upper 6.1 m of soil followed by placement in lifts and compaction would have been approximately 2.6 times the cost of the dynamic compaction.

6.2 HIGHWAY EMBANKMENT ON LANDFILL DEBRIS

Project Description

The Route 7 bypass around Manchester, Vermont crosses two areas underlain by old refuse. The southern area designated as area 1 is approximately 61 m long, and the planned embankment extended to a height of approximately 7 m above present grade. The northerly area designated as area 2 is approximately 91 m long, and the new embankment extends to heights of 3 to 3.7 m above present grade.

Subsurface Conditions

Both landfills were covered with about 0.6 m of gravelly glacial till that was used as a cover material. Below this level, old landfill material was present to depths ranging from 1 to 3.4 m but averaging about 2 m in area 1. The landfill was described as consisting of miscellaneous materials including metals, plastic, bags, glass and miscellaneous trash. No paper, food, or other biodegradable materials were encountered within the landfill. Occasional seams or layers of silty sand were encountered within the trash, but these were probably thin layers of daily cover. Standard penetration tests ranged from 10 to 14 blows for 300 mm, with some values as low as 7.

At area 2, the thickness of the trash was approximately 1 to 6 m and averaged 3 m. The trash consisted of the same classification as area 1. The water table was determined to be at a depth of 5.5 m in landfill area 2.

Both landfills were underlain by a medium-dense to-dense silty sand and gravel containing boulders and cobbles.

The age of the landfills at the time of dynamic compaction was determined to be approximately 14 to 18 years after closure. Ordinarily, this would mean the land fill was in the middle age of decomposition. However, the absence of organic materials within the trash would indicate that it was in an older stage of decomposition. Because no methane gas was noted on the boring logs it is likely that the decomposition of the highly organic materials was complete.

Design Concerns

When landfills decompose, relatively loose structure is all that remains, creating the potential for significant total and differential settlement. For this reason, some method of ground improvement was necessary, and dynamic compaction was selected to reduce the potential for this predicted movement.

Predicted Densification Procedure

Using the guidelines in this technical summary, the first step would be to calculate the tamper mass and drop height for a desired depth of improvement ranging from 3.4 m maximum in area 1 to 6 m maximum in area 2.

For area 2:

Using equation 1 and $n = 0.4$ for a landfill and $D = 6$ m

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

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

From figure 20, the energy can be obtained with tampers ranging from about 13 to 16 Mg and drop heights ranging from 14 to 30 m.

For a 14-Mg tamper,

Use $$H = \frac{224 \text{ Mgm}}{14 \text{ Mg}} = 16 \text{ m}$$

For area 1:

Using equation 1 with n = 0.4, F = 3.4 m, and W = 14 Mg

$$H = \frac{(3.4/0.4)^2}{14 \text{ Mg}} = 5.2 \text{ m}$$

The second step is calculating how much energy to apply. Using table 3 as a guide, E=600 to 1100 kJ/m^3 for a landfill. Because the SPT values indicate a medium dense condition, E = 800 kJ/m^3 was selected.

Using the average fill thickness, the required average applied energy can be calculated.

For area 2, E = (3 m) x (800 kJ/m^3) = 2400 kJ/m^2 = 2.4 MJ/m^2
For area 1, E = (2 m) x (800 kJ/m^3) = 1600 kJ/m^2 = 1.6 MJ/m^2

The third step is determining the grid spacing and number of drops. Assume all the energy can be applied in one pass. For a 14-Mg tamper, the diameter is typically 1.6 m, suggesting a grid spacing of 2.3 m. Because the highway department planned on using a surface compactor following dynamic compaction, the ironing pass was eliminated. The number of drops can now be calculated using equation 2.

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

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

N = 5.66 drops or 6 drop per grid point.

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

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

$N = 11.6$ or 12 drops per grid point.

To reduce the number of drops, the drop height could be increased because the equipment provided for area 2 will have the capacity to lift the tamper to 16 m. To have the same number of drops, six, as for area 2, use a drop height of:

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

$H = 10$ m

After dynamic compaction, the maximum SPT value according to table 2 would be anticipated to be 20 to 40.

The estimated induced settlement would be (20%)(3 m) = 0.6 m in area 2 and (20%)(2 m) = 0.4 m in area 1.

Actual Densification Procedure

Before starting dynamic compaction, the site was leveled by lowering the elevation in the high portion of the site and then placing some of the debris in the lower portion of the site. Because the debris was variable, a 0.6-m blanket of silty sandy gravel was placed on the surface as a working mat.

The specifications required a 13.6-Mg (minimum) tamper, and an 18-m drop in the shallow fill area and a 27-m drop height in the deeper fill area. The contractor used a 14-Mg tamper and elected to apply the energy in three phases. The first phase consisted of dynamic compaction on a grid basis with a spacing of 4.6 m between drop points. The second phase consisted of the same grid pattern offset from the first by 2.3 m so as to be situated between the phase, seven drops were applied. This resulted in an average energy at the ground surface of 2.5 Mg/m^2 for area 1 and 3.75 MJ/m^2 for area 2, which is considerably more energy than required by the calculations based on the presented guidelines. However, specifications required a drop height much greater than that required by equation 2, thereby resulting in more energy being applied.

Crater depths were monitored during dynamic compaction. In the first phase, the crater depths were typically 1 m. In the third phase, which took place at the same location as the first phase, the crater depths were 0.5 m. Heave measurements were taken adjacent to the drop point locations and heave was not observed. Figure 10 is a job photo illustrating the dynamic compaction equipment and the grid spacing plus crater depths.

Ground Improvement

No soil borings were made after dynamic compaction. The initial plan was to install settlement plates along the completed sections of the roadway, but this was not undertaken. Discussions with the highway engineer indicate that the pavement sections have performed well in this area and there is no evidence of settlement.

Contracting Procedure and Cost

A method specification was prepared by the agency for this project. The specification included the tamper weight, drop heights ranging from 18 m in the shallow fill areas to 27 m in the deep fill areas, plus the number of phases of energy application and the spacing between the drop point locations. The number of drops at each location was specified to range from a minimum of 6 to a maximum of 10.

A specialty contractor was awarded this project and was able to demonstrate that 7 drops were sufficient to achieve satisfactory densification. The cost for dynamic compaction was $3.30 per m^2. This cost does not include the placement of the 0.6-m gravel blanket used as a working mat.

CHAPTER 7

DYNAMIC COMPACTION COSTS

The cost of dynamic compaction will vary as a function of the depth of improvement required. A greater depth of improvement requires a heavier tamper and a higher drop height. Therefore the equipment to undertake this work also requires a much heavier lifting crane. For tampers up to 16 Mg, costs are relatively predictable because conventional lifting equipment can be used to raise and drop the tamper repeatedly. Table 4 compares costs for dynamic compaction versus the size of tamper.

Table 4. Dynamic compaction costs.

Size of tamper required (Mg)	**Unit cost Dollars/m²**
4 to 7	5.50 to 8.00
7 to 15	8.00 to 10.75
15 to 23	10.75 to 16.25
23 to 32	16.25 to 32.25
32 to 91	Negotiated for each job
Note: Prices based on projects undertaken from 1985 to 1993	

Costs for dynamic compaction vary considerably depending upon geographic location, type of deposit to be densified, and availability of local contractors. For lighter tampers (less than about 7 to 9 Mg), many excavating and earth moving contractors have adequate equipment on hand to undertake this work. When the size of the tamper is in a range 9 to 16 Mg, heavier compaction lifting and equipment are required.

When the tamper exceeds 16 to 18 Mg, specialized equipment will be necessary. This equipment could consist of a normal lifting crane modified with a large-diameter hoist for repeatedly raising and dropping the tamper. Figure 15 shows a Manitowoc 4000 crane with a special hoist on the back for repeatedly raising and dropping a 27.2-Mg tamper. Figure 16 shows a specialized crane that has been developed for lifting tampers in the range of 27.2 Mg. The costs for dynamic compaction are much higher for these specialized pieces of equipment using the heavier tampers.

For planning purposes, the desired depth of improvement is selected and then equation 1 in chapter 4 is used to predict the proper energy. After selecting a drop height, the mass of the tamper is then determined; table 4 can be used to estimate the cost. More refined methods of estimating costs are presented in FHWA-SA-95-067.

In addition to the cost for dynamic compaction, there may be other factors such as:

- The construction of a working platform over soft ground.
- Additional fill requirements to maintain the original grade.
- Undercutting of weak deposits such as Zone 3 soils that don't respond to dynamic compaction.

Though a granular blanket generally is not used on firm ground, on weak deposits such as landfills it may be necessary to import 0.3 to 0.9 m of a crushed rock or granular material. This would add to the cost for dynamic compaction. If the ground subsidence is significant and the grade must be maintained, additional fill would be required to compensate for the induced settlement which could be 5 percent to 10 percent of the thickness of the densified ground.

REFERENCES

1. J.P. Welsh (June 1986), "In-Situ Testing For Ground Modification Techniques," Use of In-Situ Tests in Geotechnical Engineering, *ASCE Geotechnical Special Publication No. 6*, pp. 322-335.

2. W. Loos (1963), "Comparative Studies of the Effectiveness of Different Methods for Compacting Cohesionless Soils," *1st International Conference on Soil Mechanics and Foundation Engineering*, Vol. 3, pp. 174-179.

3. Corps of Engineers (April 1938), "Compaction Tests and Critical Density Investigation of Cohesionless Materials for Franklin Falls Dam," *U.S. Engineer Office*, Boston, MA.

4. N.B. Hobbs (1976), Discussion of Symposium "Ground Treatment By Deep Compaction," *Institution of Civil Engineers*, London, p. 104.

5. L.M. Bobylev (6 Dec. 1963), "Distribution of Stresses, Deformations, and Density in Soil During Consolidation of Made Ground by Tamping Plates," *Osnovoniya, Fundamenty; Mekhonika Gruntov*.

6. L. Menard and Y. Broise (March 1975), "Theoretical and Practical Aspects of Dynamic Consolidation," *Geotechnique*.

7. ________ (July 1986), "Dynamic Compaction for Highway Construction," *Volume 1: Design and Construction Guidelines*, FHWA/RD-86/133.

8. Lukas, R. (1995), "Dynamic Compaction", *Geotechnical Engineering Circular 1,* FHWA, DTFH 61-93-C-00072.

9. D.E. Siskind, M.S. Stagg, J.W. Kopp and C.H. Dowding (1980), "Structure Response and Damage Produced by Ground Vibration from Surface Mine Blasting," *Bureau of Mines, Department of Investigation*, RI 8507.

10. National Cooperative Highway Research Program (1975) "Treatment of Soft Foundations for Highway Embankments," *Syntheses of Highway Practice Report 29,* TRB, Washington, DC.

MECHANICALLY STABILIZED 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-SA-96-071, August 1997.

- *Geosynthetic Design and Construction Guidelines*. FHWA HI-95-038, May 1995.

- *AASHTO Standard Specifications for Highway Bridges* (1992) Section 5.8, Division I and Section 7, Division II, including 1998 Interims.

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 geosynthetics in a strip or grid configuration; a select backfill that is chiefly characterized by a restriction on its silt/clay content; and 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

Reinforcements 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 20 years, more than 20,000 Reinforced Earth structures representing over 70 million m^2 of wall facing have been completed in 37 countries. More than 7,000 walls have been built in the United States since 1972. The highest wall constructed in the United States was on the order of 28 meters.

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 to 2.25 m^2 and generally square in shape was the facing unit of choice. More recently, larger precast units of up to 5 m^2 have been used, as have much smaller dry-cast units, generally in conjunction with geosynthetic reinforcements.

Table 1. Summary of Reinforcement and Face Panel details for selected MSE wall 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.
VSL Retained Earth VSL Corporation, 2840 Plaza Place Raleigh, NC 27612	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, 61 cm high, 200 mm thick.
Georgia Stabilized Embankment Dept. of Transportation, State of Georgia No. 2 Capitol Square Atlanta, GA 30334-1002	Rectangular grid of five 9.5 mm plain steel bars on 610 x 150 mm grid 4 bar mats per panel	Precast concrete panel; rectangular 1.83 m wide, 1.22 m high, 200 mm thick with offsets for interlocking.
Hilfiker Retaining 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.
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 by 2.5 m, 140 mm thick.
MESA Tensar Earth Technologies, Inc. 5775-B Glenridge Drive, Ste 400 Lakeside Center Atlanta, GA 30328	HDPE Geogrid	MESA Standard unit (200 mm high by 450 mm long face, 275 mm nominal depth); OR MESA high performance (190 mm high by 400 mm long face, 275 mm nominal depth).
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 by 400 mm long face, 250 mm nominal depth)
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.

[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. The highest constructed RSS structure to date has been at 1H:1V to 33.5 m.

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

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 of MSE retaining wall with precast facing are constructed on average every year in the United States, which may represent more than two-thirds 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 a 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, however, 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, to form a wall system referred to as modular block wall (MBW). It has been reported that more than 300 such structures have been constructed in the United States for highway applications to date. The current yearly usage for transportation-related applications is estimated at about 50 projects per year.

Table 2. Representative list of Geotextile and Geogrid manufacturers and suppliers.[1]

Akzo Nobel Industrial Systems Ridgefield Business Center Suite 318, Ridgefield Court Asheville, NC 28802	Amoco Fabrics and Fibers Co. 900 Circle 75 Parkway, Suite 300 Atlanta, GA 30339	Bayex Inc. 14770 East Ave. P.O. Box 390 Albion, NY 14411-039
Carthage Mills 4243 Hunt Road Cincinnati, OH 45242	Hoechst Celanese Corp. P.O. Box 5650 I-85 & Road 57 Spartanburg, SC 29304	Huesker, Inc. 11107 A S. Commerce Blvd. Charlotte,, NC 28241
LINQ Industrial Fabrics, Inc. 2550 West 5th North Street Summerville, SC 29483	Nicolon Corporation 3500 Parkway Lane, Suite 500 Norcross, GA 30092	Reemay, Inc. 70 Old Hickory Blvd. Old Hickory, TN 37138
Spartan Technologies P.O. Box 1658 Spartanburg, SC 29304	Strata Systems, Inc. 425 Trible Gap Road Cummings, GA 30130	Synthetic Industries Cons;truction Products Division 4019 Industry Drive Chattanooga, TN 37416
Tenax Corporation 4800 East Monument Street Baltimore, MD 21205	Tensar Earth Technologies 5775-B Glenridge Drive Suite 450, Lakeside Center Atlanta, GA 30328	Wellman, Inc. 2748 Tanager Ave. Commerce, CA 90040

[1] List is from the Industrial Fabrics Association International, Geotextile and Geomembrane Divisions membership list.

The use of RSS structures has expanded dramatically in the last decade, and it is estimated that several thousand RSS structures have been constructed worldwide. Currently, 50 to 70 RSS projects are being constructed annually in connection with transportation related projects, with an estimated projected vertical face area of 100,000 m^2.

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 indicate 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-SA-96-071 *Mechanically Stabilized Earth Walls and Reinforced Soil Slopes, Design and Construction Guidelines*.

CHAPTER 2

DESIGN CONSIDERATIONS

2.1 APPLICATIONS

MSE 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 percent 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.

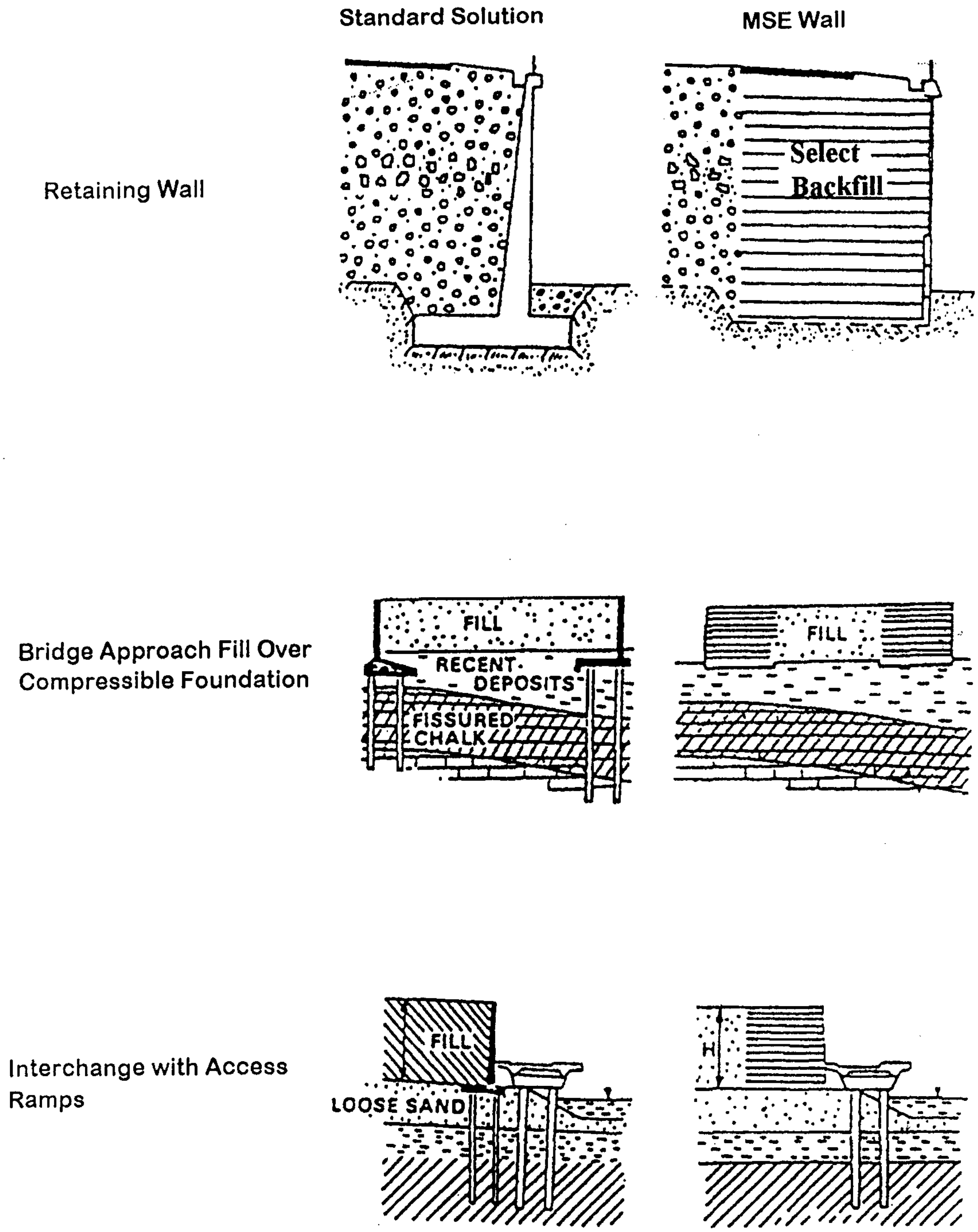


Figure 1. MSE wall. Urban applications.

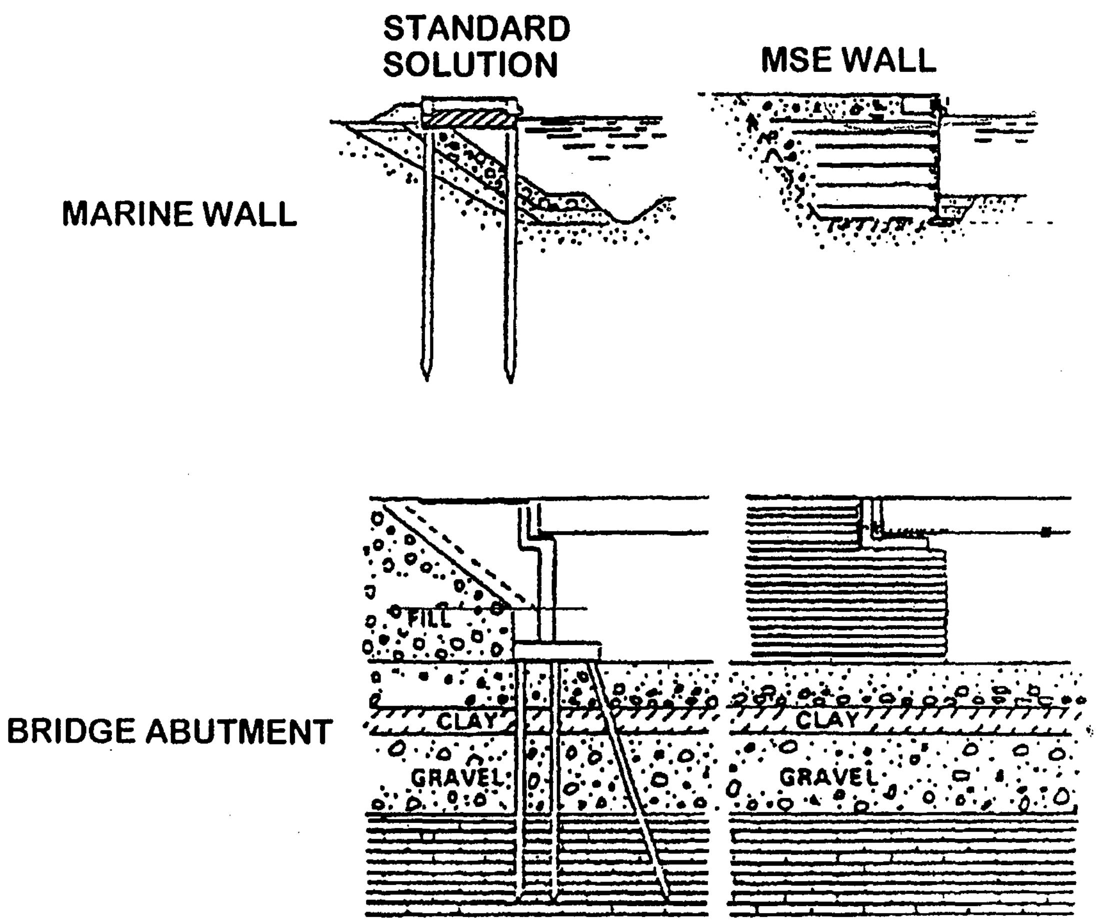


Figure 2. MSE wall applications, abutments.

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 usually 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.

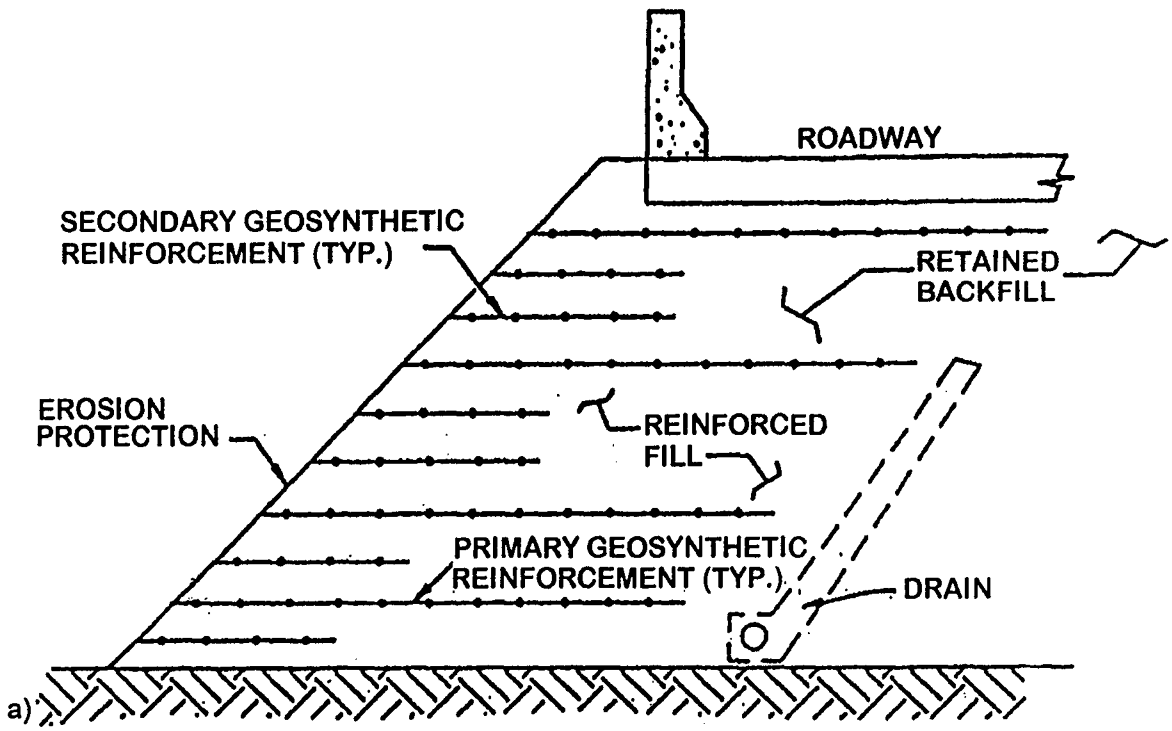
- To increase the stability of the slope, particularly if a steeper than *safe* unreinforced slope is desirable or after a failure has occurred as shown in figure 3a.

- 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 edges 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 though the use of geosynthetics with in-plane drainage capabilities (e.g., nonwoven geotextiles) that allow for rapid pore pressure dissipation in the compacted soil.



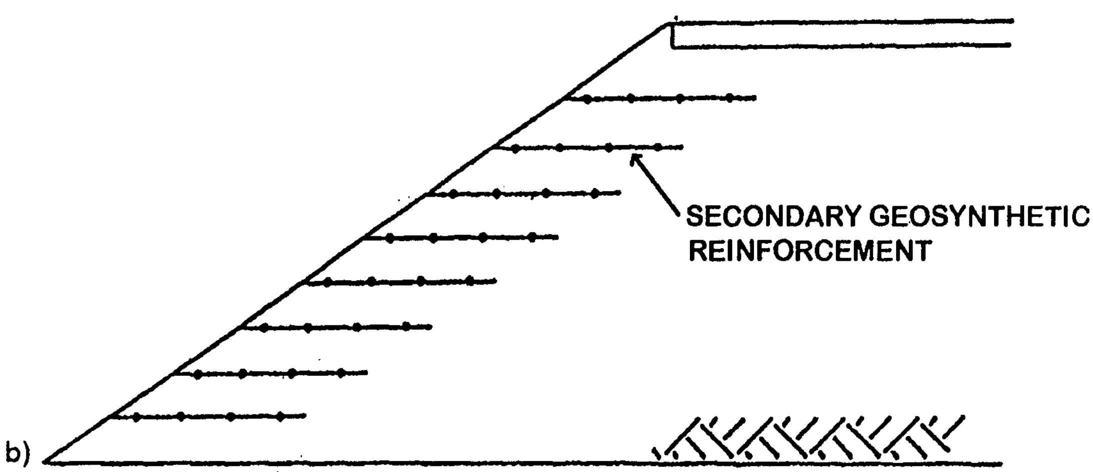


Figure 3. Slope reinforcement using geosynthetics to provide slope stability.

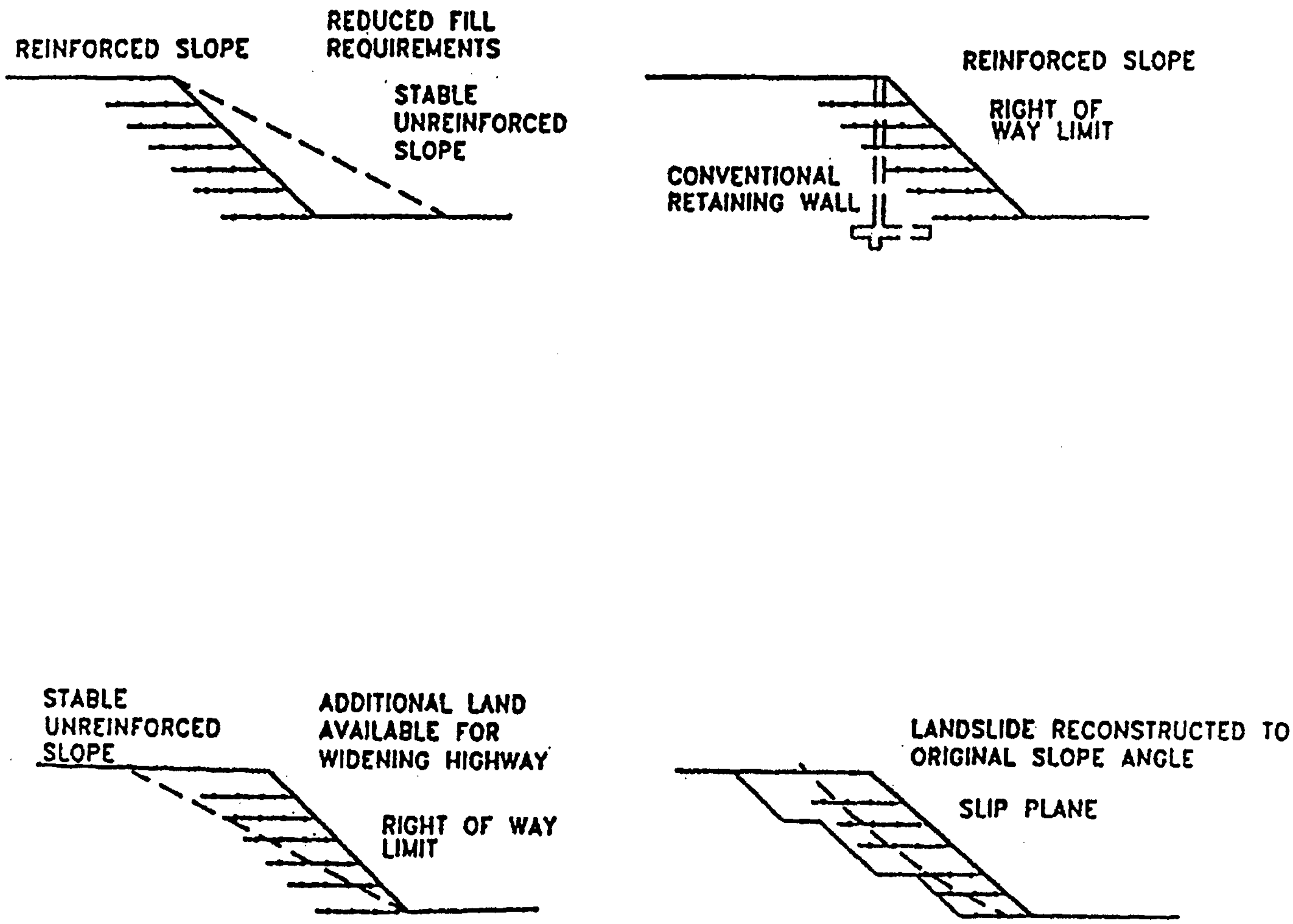


Figure 4. Application of reinforced soil slopes.

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.
- 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. 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.

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 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. Reinforced Soil Slopes (RSS)

The economic advantages of constructing a safe, steeper RSS than would normally be possible are the resulting material and rights-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 maintenance issues such as mowing grass-faced, steep slopes that must be addressed.

In terms of performance, due to inherent conservatism in their design, RSS are actually safer than flatter slopes designed at 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.

Disadvantages

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

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

- MSE 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 factors that influence the selection of an MSE alternative for any project include:

- Environmental conditions.

- Geologic and topographic conditions.

- Size and nature of the structure.
- Aesthetics.
- Durability considerations.
- Performance criteria.
- Availability of materials.
- Experience with a particular system or application.
- Relative cost.

Many MSE wall 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 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. On most projects, it is likely that a number of systems may be determined to be appropriate.

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.

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 these latter conditions, 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.

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 transverse 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 MSE structures 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.

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 4. 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, typically highly acidic regimes, or for polyesters (PET) highly alkaline regimes.

A secondary environmental issue is site accessibility, which may dictate the nature and size of the facing for MSE construction. Sites with poor accessibility or remote locations may lend themselves to lightweight facings such as metal skins; modular blocks (MBW) which could be erected without heavy lifting equipment; or the use of geotextile 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 an MSE that can be constructed. Structures in excess of 25 m have been successfully constructed with steel reinforcements although such heights for transportation-related structures are rare. RSS embankments have been constructed to equal 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 the area of footing necessary to support the beams. Since the allowable bearing capacity in the reinforced fill is usually limited to 200 KPa, 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 to 4 meters are often uneconomical, as the cost of the overturning moment leg of the traffic barrier adds up to one-third to 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.

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 percent of the facing cost, which on average would mean a 2-to-4 percent 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) where the small size introduces savings in erection equipment cost and the need to cast special, made-to-order concrete panels to fit what is often irregular geometry. MBW facings may be manufactured in color and with a wide variety of surface finishes.

The outward face treatment of RSS, generally by vegetation, 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 perpendicular 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.

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

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.

Where significant differential settlements perpendicular to the wall face are anticipated, the reinforcement connection must allow for vertical movement or the reinforcement placed on a sloping fill surface that is higher at the back end of the reinforcement to compensate for the greater vertical settlement. This latter construction technique, however, requires that surface drainage be carefully controlled after each day's construction.

2.4 LIMITATIONS

The 1997 *AASHTO Interim Specifications for Highway Bridges*, indicates that MSE walls should not be used under:

- When utilities other than highway drainage must be constructed within the reinforced zone.

- 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.

Similar restrictions should be considered for RSS structures.

CHAPTER 3

CONSTRUCTION SYSTEMS, MATERIALS, AND METHODS

Since the expiration of the fundamental process, reinforcement and concrete facing panel patents obtained by the Reinforced Earth Co. for MSE wall 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 MSE 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

MSE systems can be described by the reinforcement geometry, stress transfer mechanism, reinforcement material, extensibility of the reinforcement material, and the type of facing and connections.

System Type

Two functional types can be considered:

- MSE retaining walls characterized by a vertical or near vertical facing, usually with a precast of drycast concrete panel for permanent applications.

- RSS embankments characterized by an inclined face between 35 and 70 degrees, 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, and coated geosynthetic strips over a load-carrying fiber.

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

- **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.

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.

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 drainage paths 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 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 segmental blocks (MBW) units**. These are relatively small, squat concrete units that have been specially designed and manufactured for retaining wall applications. The weight of these units commonly ranges from 15 to 50 kg, with units of 35 to 50 kg routinely used for highway projects. Unit heights typically range from 100 to 200 mm for the various manufacturers. Exposed face length usually varies from 200 to 450 mm. Nominal width (dimension perpendicular to the wall face) of units typically ranges between 200 and 600 mm. 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®, Versalock, Allen 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 usually 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.

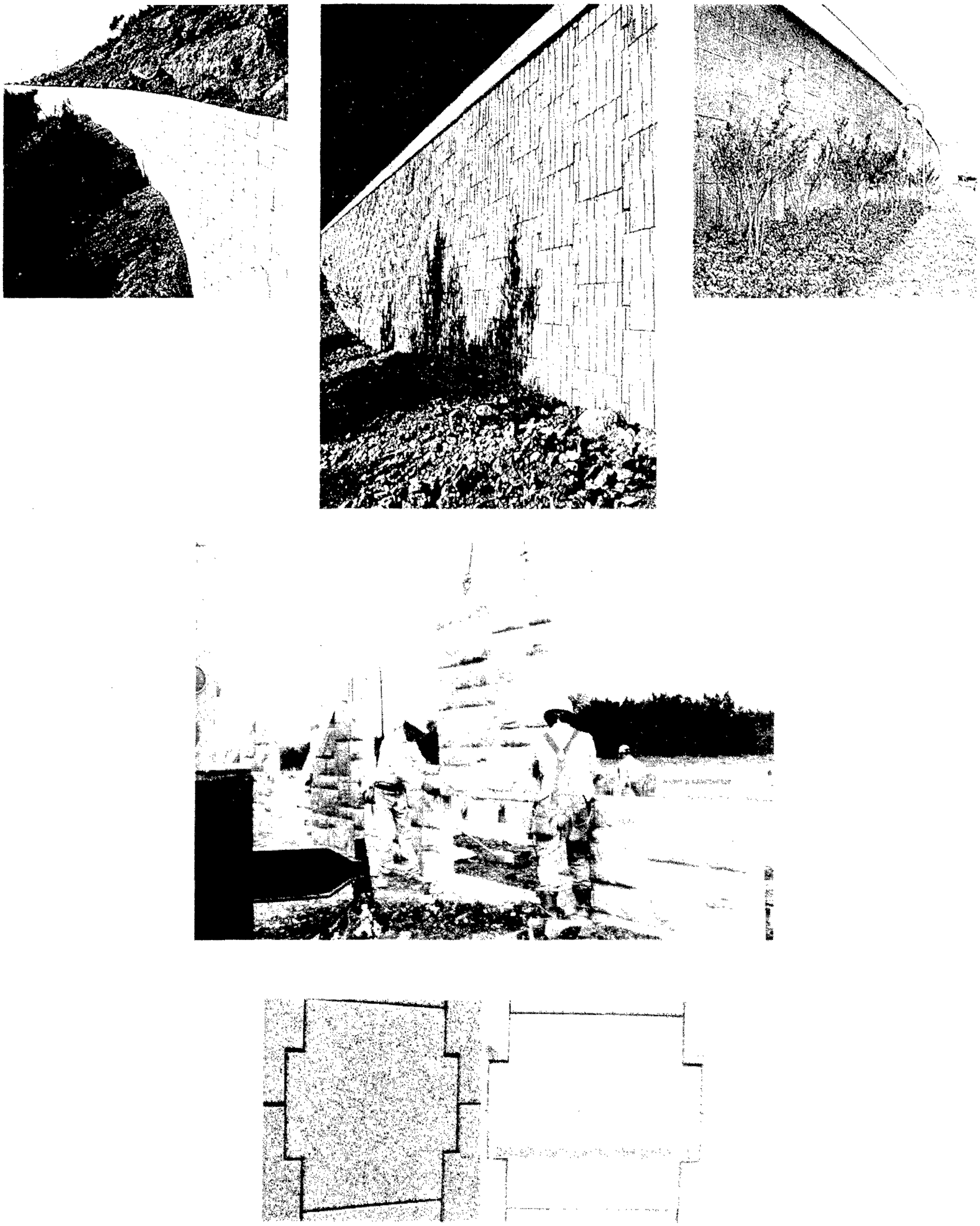


Figure 5. MSE wall surface textures.

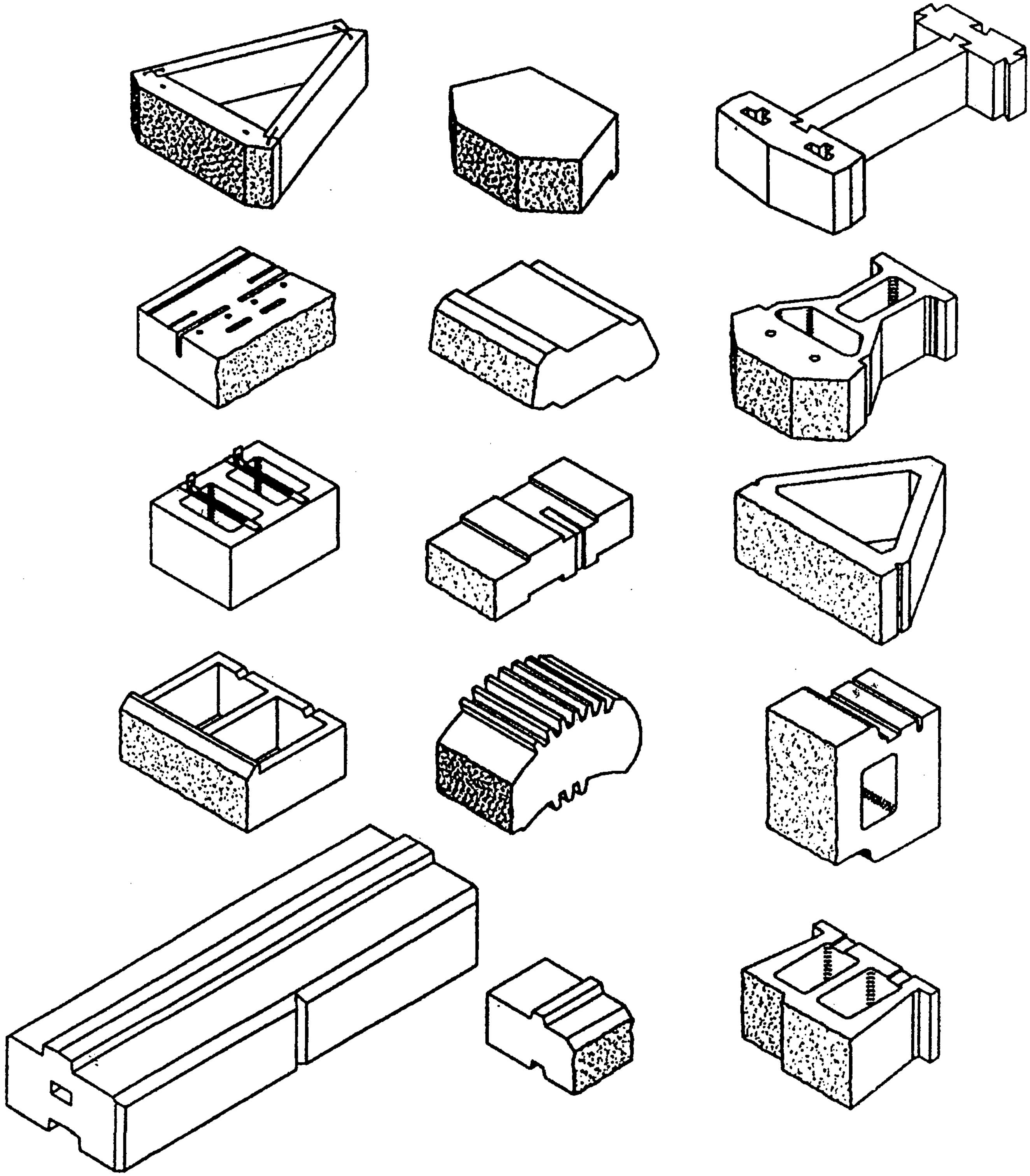


Figure 6. Examples of commercially available MBW units (from Design Manual for Segmental Retaining Walls).

- **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 (e.g. target practice) and damage due to fire. Alternately, a geosynthetic grid used for the reinforcement of the soil 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.

- **Postconstruction Facing** - For wrapped faced walls, the facing - whether geotextile, geogrid, or wire mesh can be attached after construction of the wall by shotcreting, guniting, or attaching prefabricated facing panels made of concrete, wood, or other materials. This 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 and blend aesthetically into the environment. Retaining structures using precast concrete elements as the facings can have surface finishes similar to any reinforced concrete structure.

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

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.

Recently introduced dry cast segmental block MBW facings raise some concerns as to their durability in aggressive freeze-thaw environments because their water absorption capacity can be significantly higher than that of wet-cast concrete. Historical data provide little insight as their usage history is less than a decade. 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 become an aggressive aging media for some geosynthetic products potentially used as reinforcements. Freeze-thaw durability is enhanced for products produced at higher compressive strengths low water absorption ratios, and/or sprayed with a posterection sealant.

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. For steeper slopes, armoring is necessary. It may consist of:

- Geosynthetic Erosion Control Mats.
- Riprap, fabric formed concrete, gunite.
- Structural elements used for MSE walls.

3.3 REINFORCED BACKFILL MATERIALS

MSE Structures

MSE walls require high quality backfill for durability, good drainage, constructability, and high 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 generally eliminate soils with high clay contents.

From a reinforcement capacity point of view, lower quality backfills could be used for MSE 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 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.**

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

Plasticity Index (PI) shall not exceed 6.

[a]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 20 mm 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 standard tests (or an equivalent sodium sulfate value) of less than 30 percent 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 above and below optimum. The compaction requirements of backfill are different in close proximity to the wall facing (within 1.5 to 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.

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 limits are shown in table 4 as follows:.

Table 4. Recommended 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

Reinforced fill soils must meet the indicated criteria to be qualified for use in MSE 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 5. Recommended 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

The reinforced fill criteria outlined represent materials that have been successfully used throughout the United States and resulted in excellent structure performance. 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 to 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.

RSS Structures

Reinforced Soil Slopes are normally not constructed with rigid facing elements. Slopes constructed with a flexible face can thus readily tolerate minor distortions that could result from settlement, freezing and thawing, or wet-drying of the backfill. As a result, any soil meeting the requirements for embankment construction could be used in a reinforced slope system. However, a higher quality material offers fewer durability concerns for the reinforcement and is easier to handle, place, and compact, which speed up construction. Therefore the following guidelines are provided as recommended backfill requirements for RSS construction:

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

Plasticity Index (PI) $\leq$ 20 (AASHTO T-90)

Soundness: Magnesium sulfate soundness loss less than 30 percent after 4 cycles, based on AASHTO T-104.

* 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.

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

Fill materials outside of these gradation and plasticity index requirements have been used successfully; however, long-term (> 5 years) performance field data is not available. Performance monitoring is recommended if backfill soils fall outside of the requirements listed above.

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. Again, significant economy could be achieved if laboratory direct shear or triaxial test results on the proposed fill are performed, justifying a higher value.

3.4 MISCELLANEOUS MATERIALS OF CONSTRUCTION

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

All joints are covered with a geotextile strip to prevent the migration of fines from the backfill. Vertical joints, if large, may be filled in addition with a synthetic foam. The compressibility of the horizontal joint material should be a function of the wall height. Walls with heights greater than 15 m 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.

3.5 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 MSE wall systems with precast facings

The construction of MSE 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 constructed 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 and 150 mm 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 facing panels may be full, or half-height panels, depending upon the type of facing are used. The first tier of panels must be braced up to maintain stability and alignment. For construction with modular dry-cast blocks, full sized blocks are used throughout with no bracing.

  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 to 100 percent of AASHTO T-99 maximum density and within the specified range of optimum moisture content.

  A key to good performance is *consistent* placement and compaction. 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. 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 fill has been brought up to the level of the connection they are generally placed perpendicular to the back of the facing panels.

- **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:

1. Placing the soil.

2. Placing the reinforcement.

3. Constructing the face.

The usual construction sequence is shown in figure 10.

- **Site Preparation**. Consists of successively clearing and grubbing, levelling 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 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.

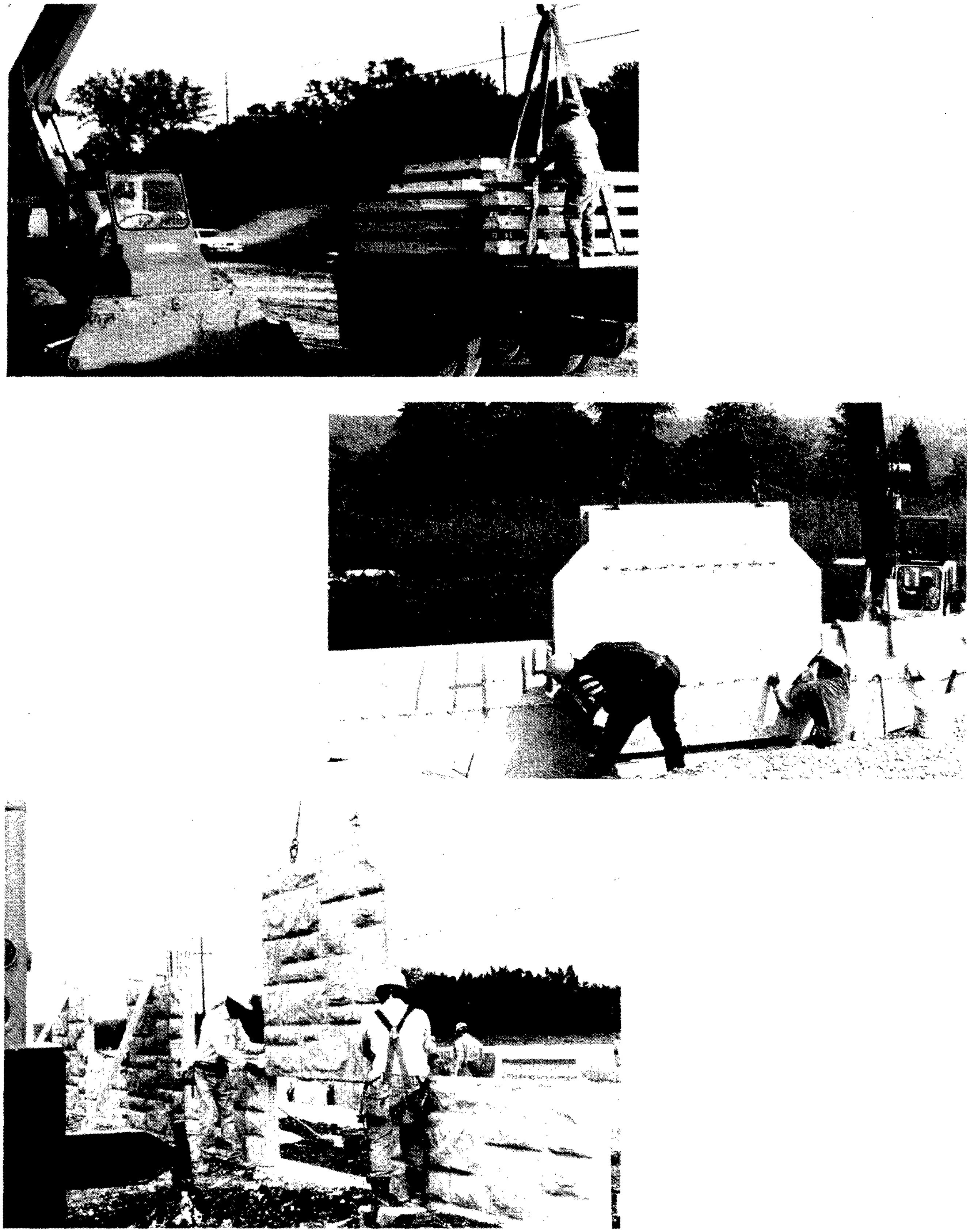


Figure 7. Erection of precast panels.

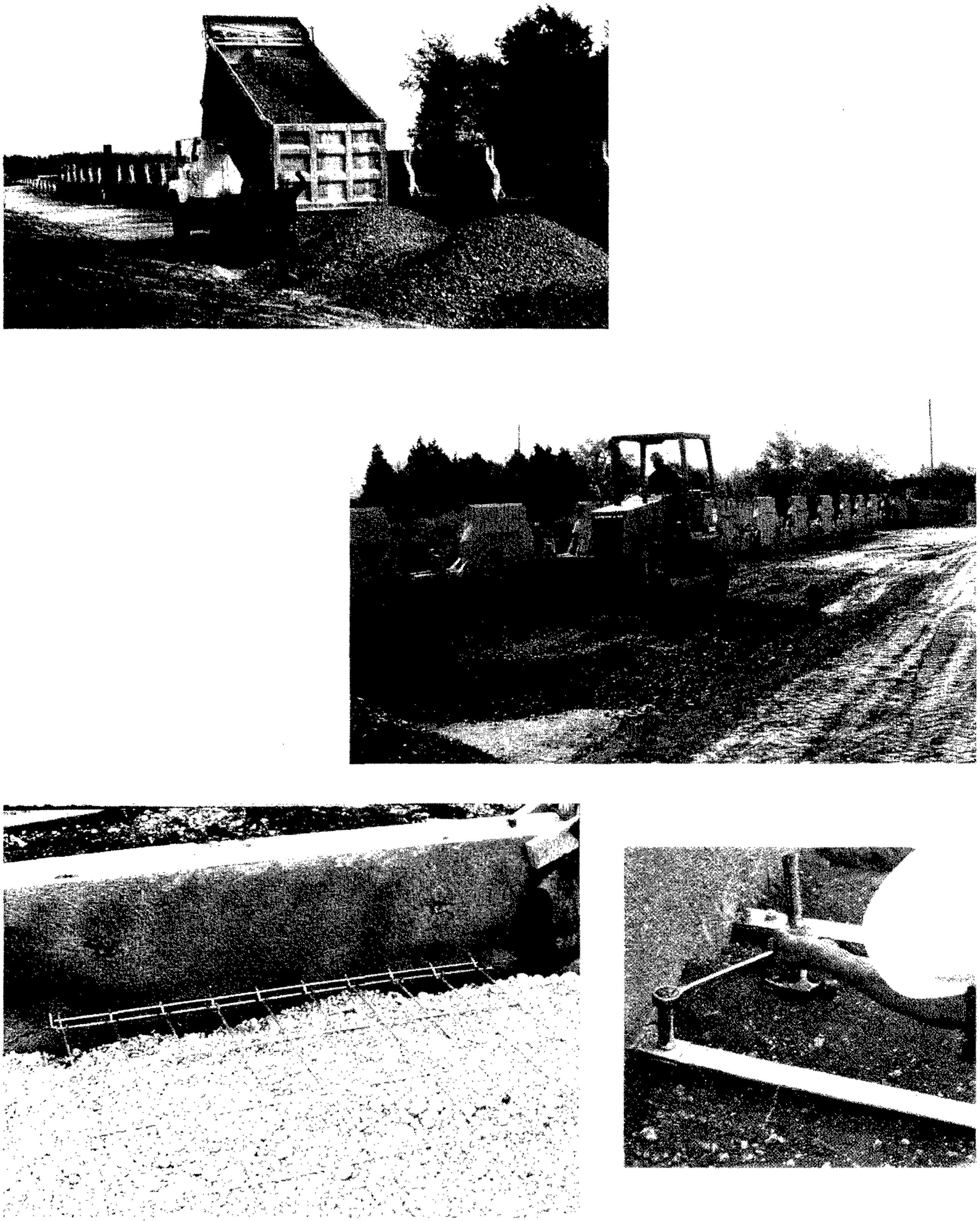


Figure 8. Fill spreading and reinforcement connection.

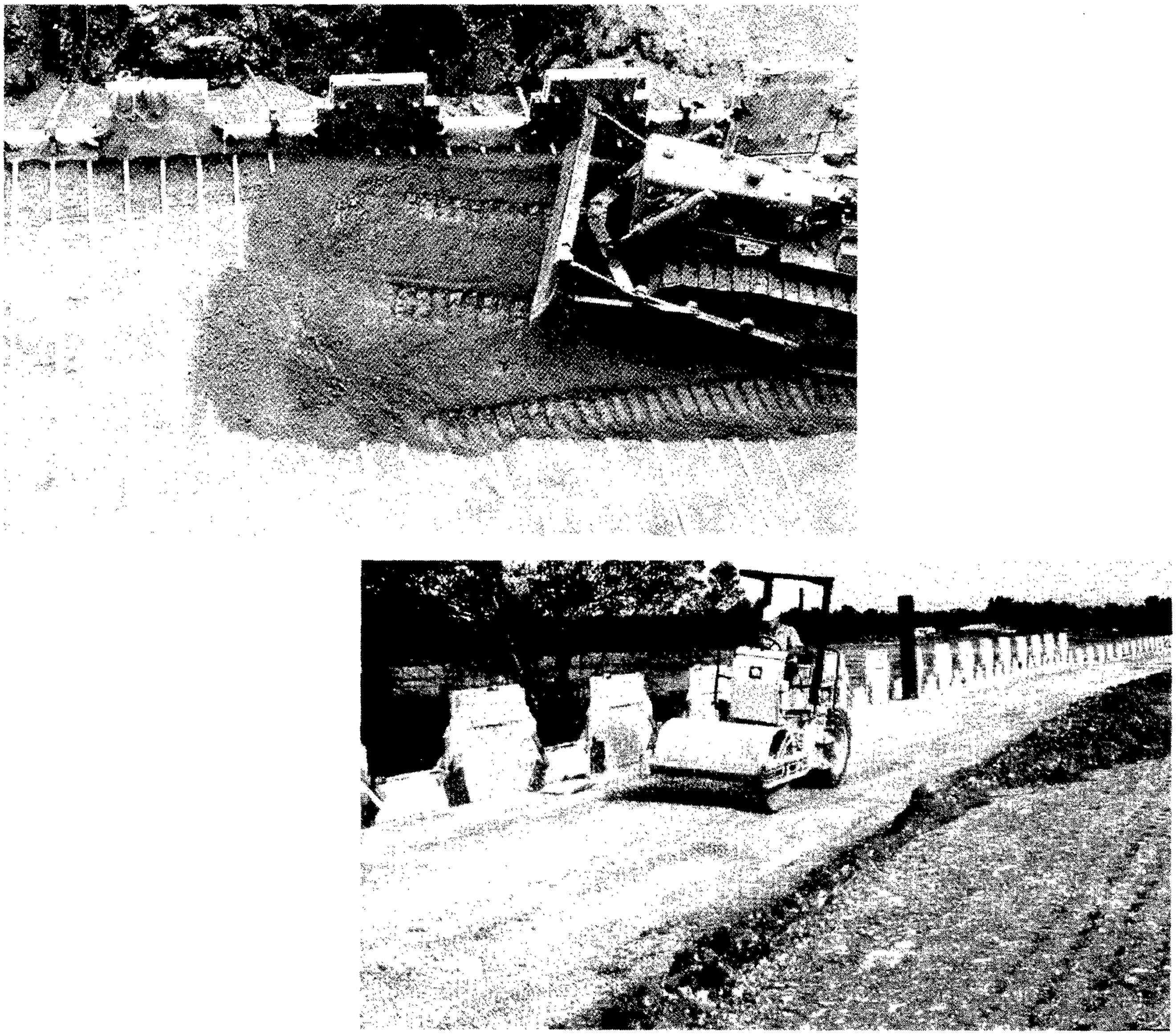


Figure 9. Compaction of backfill.

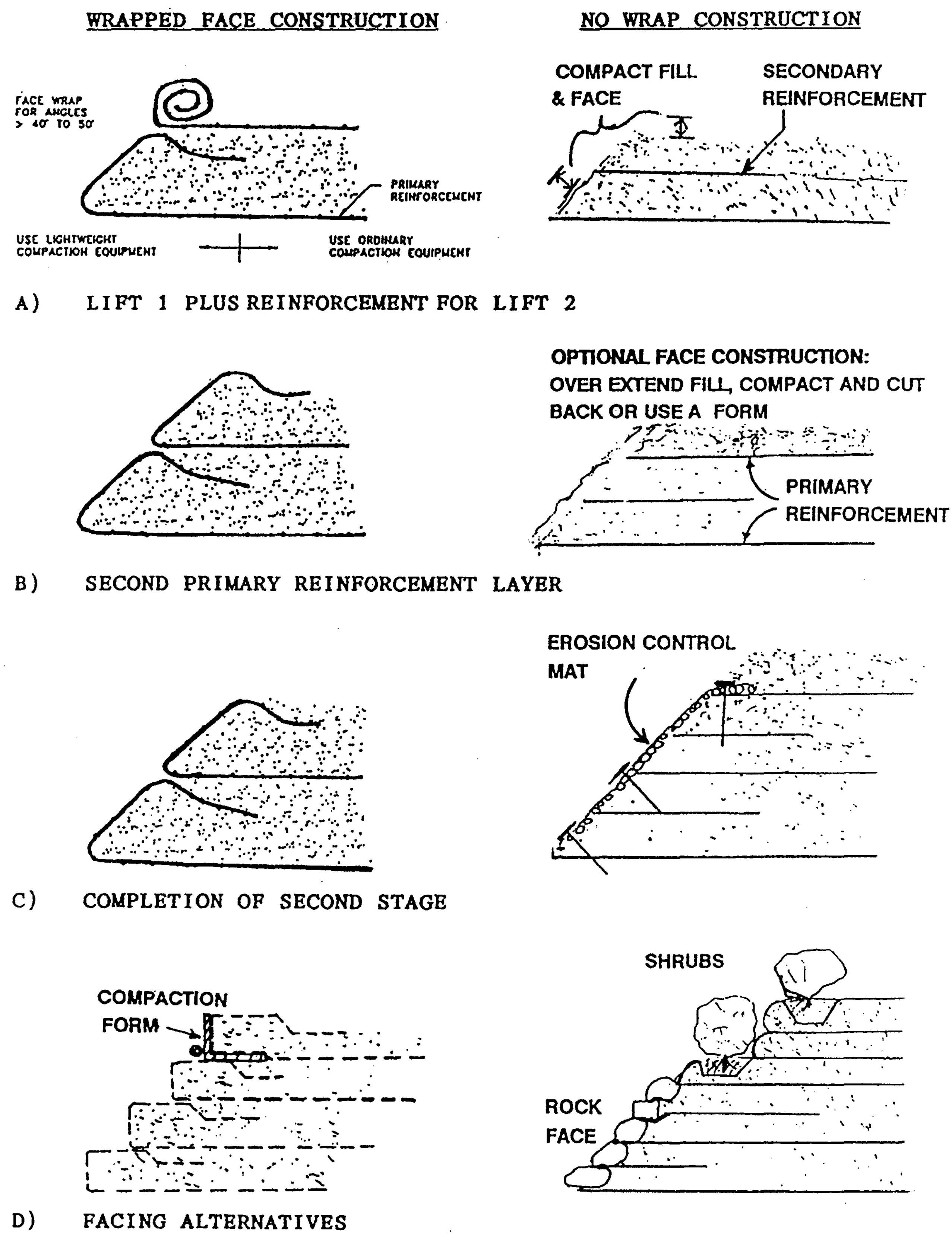


Figure 10. Construction of reinforced soil slopes.

- **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 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 vehicle or smooth drum roller for cohesive materials. Provide close control on the water content and density of the backfill. It should be compacted at least 95 percent of the standard AASHTO T99 maximum density within 2 percent of optimum moisture.

- **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 is not 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). In this case, the reinforcement can be simply extended to the face. For this option, a facing treatment 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 into the embankment below the next reinforcement layer (see figure 10).

- For steep slopes, form work may be required to support the face during construction, especially if lift thicknesses of 450 to 600 mm 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-SA-96-071 *Mechanically Stabilized Earth Walls and Reinforced Soil Slopes Design and Construction Guidelines.*

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 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 two mechanisms: friction and passive resistance.

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, and geotextile 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 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.

Mode of Reinforcement Action

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.

Tension is the most common mode of action of tensile reinforcements. All "longitudinal" reinforcing elements (i.e., reinforcing elements aligned in the direction of soil extension) are generally subjected to high tensile stresses. Tensile stresses are also developed in flexible reinforcements that cross shear planes.

Shear and Bending. "Transverse" reinforcing elements that have some rigidity can withstand shear stress and bending moments.

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 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.

For most projects, it is reasonable to use well developed semi-empirical relationships in conjunction with the standard specifications for backfill to provide a conservative evaluation of pullout resistance.

For steel ribbed reinforcement, the friction factor μ^* is commonly taken as:

$$F^* = \mu^* = 1.2 + \log C_u$$
$$F^* = \mu^* = 1.5 \text{ to } 2 \text{ at the top of the structure}$$
$$F^* = \mu^* = \tan\phi \text{ at a depth of 6 m}$$

with C_u being the uniformity coefficient of the backfill. When fine uniform sands are used the maximum μ^* is generally limited to 1.2.

For steel grid reinforcements with spacing between longitudinal and transverse bars of at least 150 mm, the total Pullout Resistance Factor F* is a function of an empirical Bearing Capacity Factor (F_q), and an area geometric factor α_B, as follows:

$$F^* = F_p\alpha_B = 40\left[\frac{t}{2\,S_t}\right] \text{ at the top of the structure}$$

$$F^* = N_q\alpha_B = 15\left[\frac{t}{2\,S_t}\right] \text{ at a depth of 6 m}$$

where t is the thickness of the transverse bar and S_t the longitudinal spacing between transverse bars.

For geosynthetic sheet reinforcement, the friction μ^* is commonly taken as:

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

For geosynthetic grid reinforcement, the friction factor μ^* often referred as an Interaction Factor (C_i) is commonly taken as:

$$F^* = \mu^* = 0.8 \tan \phi$$

For all reinforcements test data is required to substantiate a higher value.

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. 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

The strength of the reinforcement to resist the developed tension in the reinforcements is a function of reinforcement type.

The allowable tensile force per unit width of metallic reinforcement T_a is obtained from

$$T_a = \frac{F_y A_c}{b} FS \qquad (1)$$

where:

FS = 0.55 for strips and 0.48 for grids with rrigid facing elements. (0.55 may be used for grids with flexible facing)

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. (75 to 100 years)

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.**

Corrosion Rates - mildly corrosive backfill

For zinc

15 μm/year (first 2 years)

4 μm/year (thereafter)

For residual carbon steel

12 μm/year (thereafter)

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 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 preliminary design or for applications defined by the user as not having severe consequences should poor performance or failure occur, the allowable tensile strength T_a, may be evaluated without product specific data as:

$$T_a = \frac{T_{ULT}}{7 \cdot FS} \tag{2}$$

with T_{ULT} obtained from wide width tensile tests (ASTM D 4595).

The total reduction factor of 7 has been established by multiplying lower bound partial reduction factors obtained from currently available test data.

Further, this reduction factor should be limited to projects where the project environment meets the following requirements:

- Granular soils (sands, gravels) used in the reinforced volume.

- $4.5 \leq pH \leq 9$.
- Maximum backfill particle size of 20 mm.
- Maximum MSEW height is 10 m and
- Maximum RSS height is 15 m.

It should be noted that the total Reduction Factor may be reduced significantly with appropriate test data. It is not uncommon for products with creep, installation damage, and aging data to develop total Reduction Factors in the range of 3 to 6.

For greater details in developing project specific Reduction Factors, refer to FHWA-SA-96-072, *Corrosion/Degradation of Soil Reinforcements for Mechanically Stabilized Earth Walls and Reinforced Soil Slopes.*

4.4 DESIGN METHODS FOR MSE WALLS

Since the development of soil reinforcement concepts and their application to MSE 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.
- In some cases, the critical slip surface is partially outside and partially inside the stabilized soil mass, and a combined external/internal stability analysis may be required.

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** assumes driving forces 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 ranges from a multiple of K_a to an active earth pressure state, K_a.

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 the depth from the surface. The results from these efforts have been synthesized in a *simplified coherent gravity method*, that represents the state-of-the art 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 geosynthetics. For internal stability, a Rankine failure surface is considered because the extensible reinforcements can elongate more than the soil before failing.

4.5 SIZING FOR EXTERNAL STABILITY

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

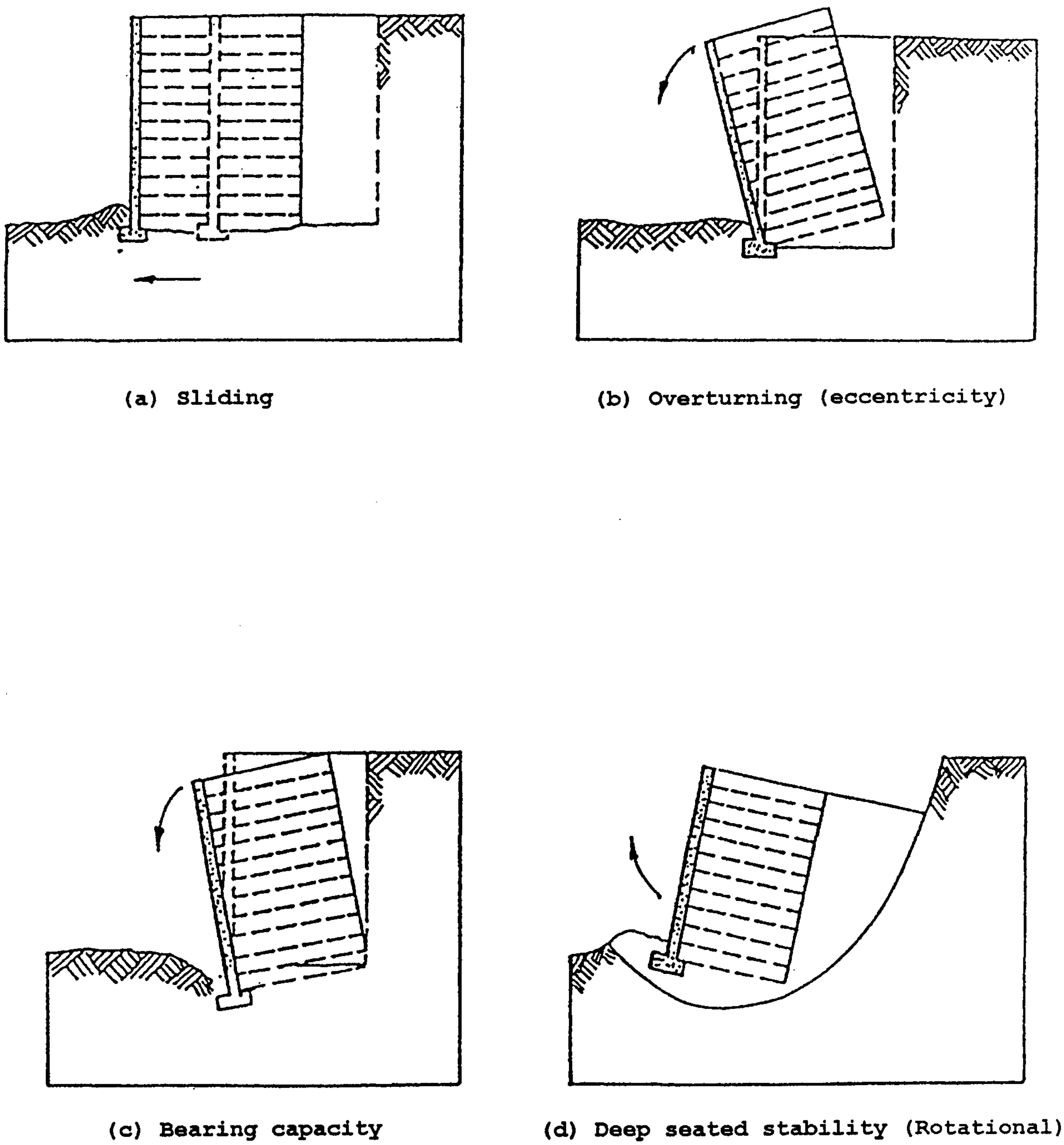


Figure 11. Potential external failure mechanisms for a MSE wall.

- 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).

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 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 percent of the height, but at least 2.5 m 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 on figure 12.

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

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

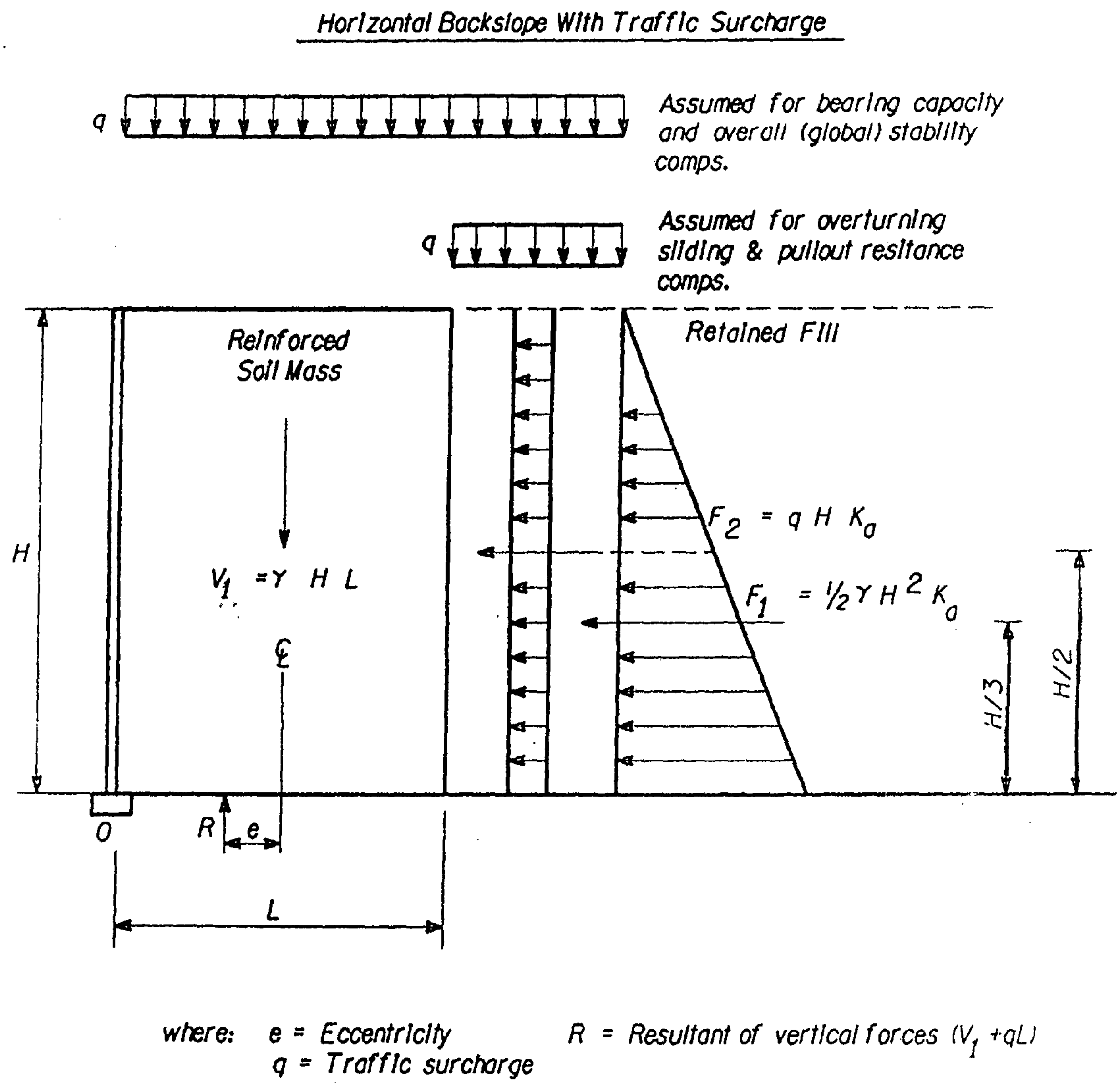


Figure 12. External analysis: earth pressures/eccentricity. Horizontal backslope with traffic surcharge.

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:

$$\sigma_v \leq q_a = \frac{q_{ult}}{FS} = \frac{q_{ult}}{2 or 2.5} \qquad (5)$$

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

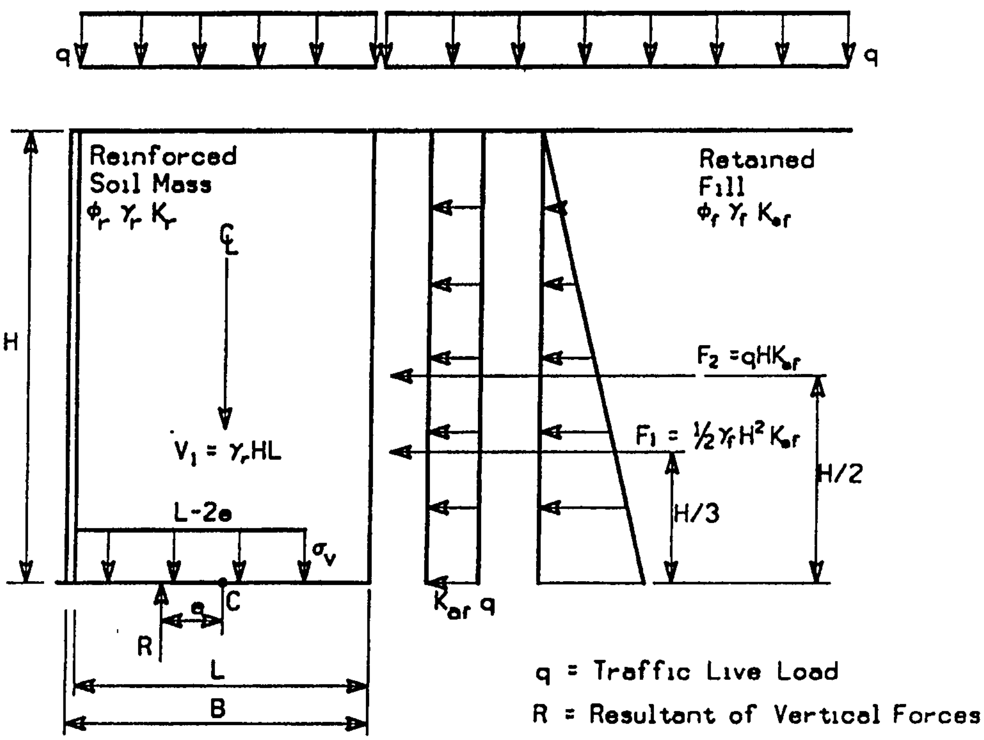
For sloping surcharge conditions, concentrated loads, seismic loads, or other unusual loading or geometric conditions, refer to FHWA-SA-96-071, *Mechanically Stabilized Earth Walls and Reinforced Soil Slopes, Design and Construction Guidelines.*

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.



SUMMING MOMENTS ABOUT POINT C:

$$e = \frac{F_1\,(H/3) + F_2\,(H/2)}{V_1 + qL} \qquad \sigma_v = \frac{V_1 + qL}{L-2e}$$

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

This maximum tensile forces line 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 ratio decreases from the top of wall to a constant value below 6 m.

This graphical figure was prepared by back analysis of the lateral stress ratio K from available field data. 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.

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 $\Delta\sigma_h$.

$$\sigma_h = K\,(\gamma_r Z + q + \Delta\sigma_v) + \Delta\sigma_h \qquad (6)$$

where: K = K(z) is shown in figure 15 and Z is the depth below the top of wall.

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.

The maximum tension, T_{max} in each reinforcement layer per unit width can therefore be calculated from:

$$T_{\max} = \sigma_h\, S_v \qquad (7)$$

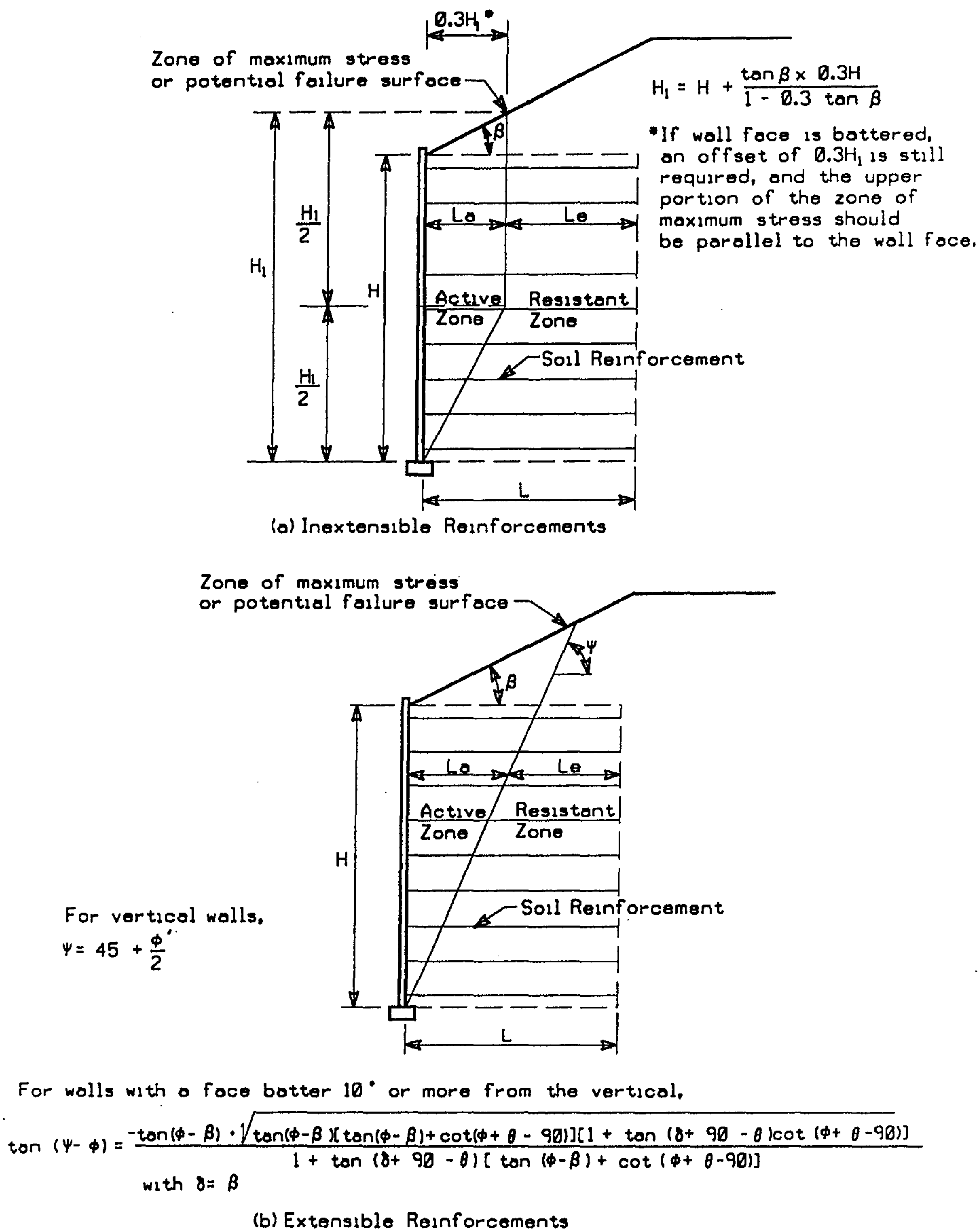
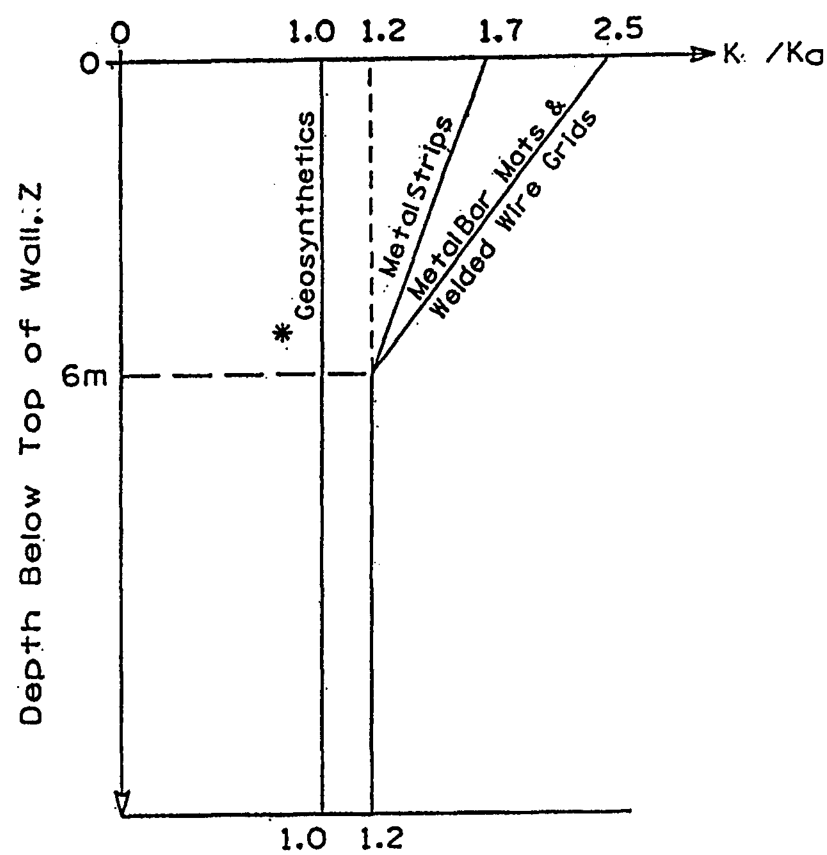


Figure 14. Location of potential failure surface for internal stability design of MSE walls.



*Does not include polymer strip reinforcement

Figure 15. Variation of the stress ratio with depth in a MSE wall.

where S_v is the vertical spacing of the reinforcement in the structure and the internal stability with respect to breakage determined from:

$$T_{max} \leq T_a R_c = T_a \frac{b}{S_h} \tag{8}$$

where R_c is the coverage ratio in which b is the gross width of the reinforcement and S_h is the center to center horizontal spacing between reinforcements.

c. Internal Stability With Respect to Pullout Failure

Stability with respect to pullout of the reinforcements requires that sufficient length (L_e) be available beyond the potential failure surface to ensure stability as follows:

$$L_e \geq \frac{1.5 \ T_{max}}{2 \ F^* \ \gamma Z' \ R_c} \geq 1 \ m \tag{9}$$

where F^* is the pullout resistance factor and $\gamma Z'$ the overburden pressure including distributed *dead load* surcharges.

4.7 DESIGN OF REINFORCED STEEPENED SLOPES (RSS)

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.

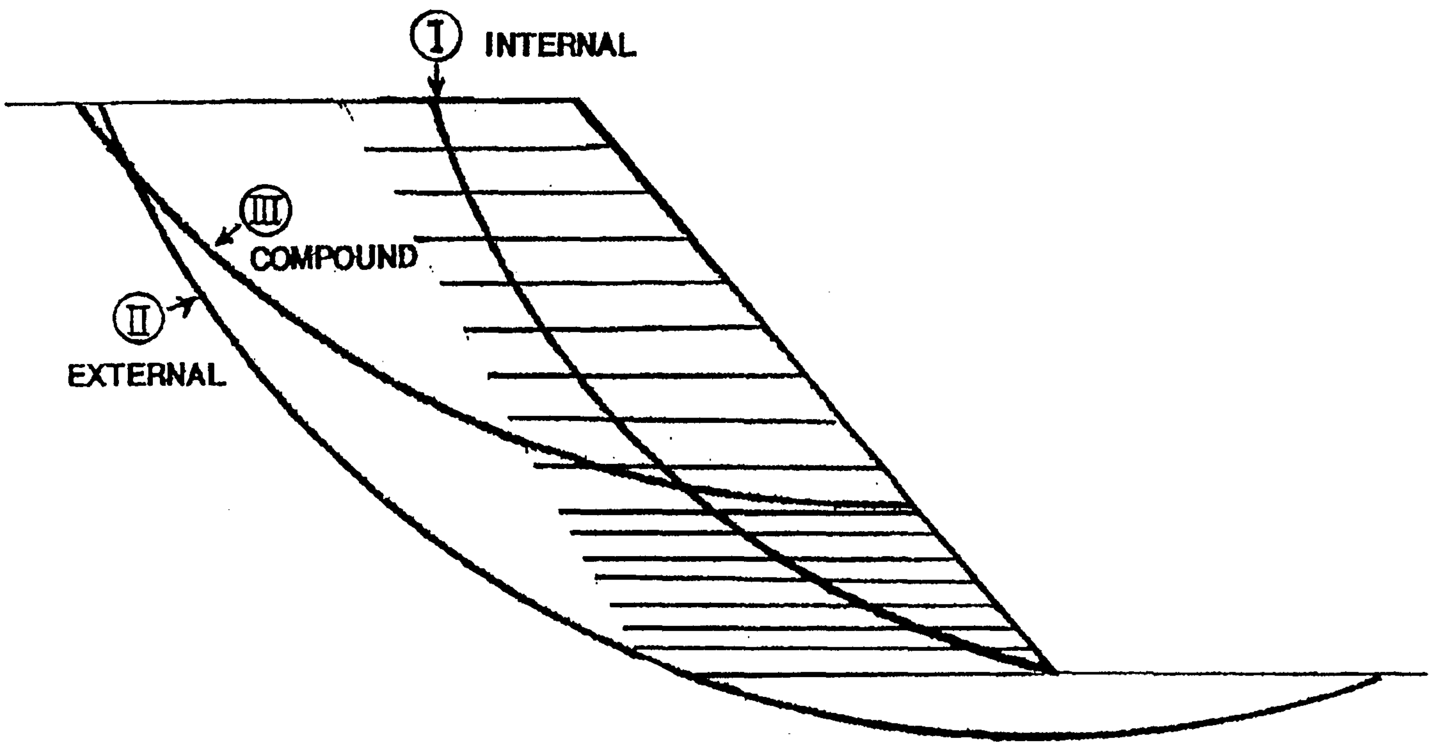


Figure 16. Failure modes for reinforced soil slopes.

- Compound, where the failure surface passes behind and through the reinforced soil mass.

In many cases, the calculated stability safety factor will be approximately equal in two or all three modes.[3]

For steepened reinforced slopes 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 total reinforcement tension T_s, required to obtain the desired factor of safety for each potential failure circle inside the critical zone is determined from:

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

where:

T_s = sum of required tensile force per unit width of reinforcement 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 the failure circle, D = radius of circle R;

FS_R = target minimum safety factor; and

FS_u = unreinforced slope safety factor.

- 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.

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 slope stability factor of safety is taken from the unreinforced failure surface requiring the maximum reinforcement. 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.

Several approaches are 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).

a. 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.

Several 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 or layout. Most of the programs are limited to simple soil profiles and, in some cases, the reinforcement layouts. Also, external stability evaluation is generally limited to specific soil and reinforcement conditions. In some cases, the programs are reinforcement-specific. These programs could be used to provide a preliminary evaluation or to check a detailed analysis. Examples include:

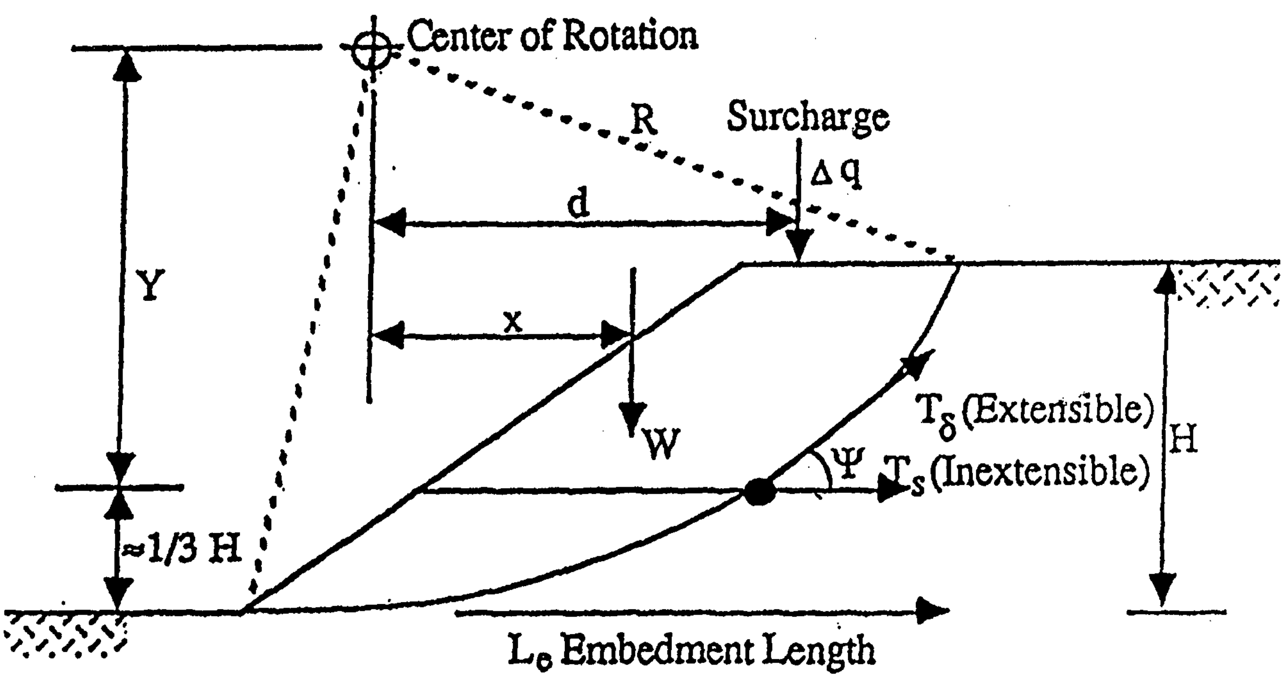


Figure 17. Modified limit equilibrium analysis for reinforced slope design.

PCSTABL6	-	Purdue University
STABGM	-	Virginia Tech
XSTABL	-	University of Idaho
UTEXAS2	-	US Army COE
UTEXAS3	-	University of Texas
New Janbu	-	Tensar
Strata Slope	-	Strata Systems
Tenslo1	-	Tensar
RSS	-	FHWA

The generic program developed by FHWA (RSS) for both reinforcement design and evaluation of almost any condition is available.

b. 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.

The design procedure in using the charts is as follows:

- Determine $T_{s\text{-}max}$ using figure 18 part A.
- Determine L_T and L_B using figure 18 part B.
- Determine the distribution of reinforcement as follows:
  - For low slopes ($H \leq 6$m) assume a uniform reinforcement distribution and use $T_{s\text{-}max}$ to determine spacing or the design tension T_d 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 $T_{s\text{-}max}$ in each zone for spacing or design tension requirements.

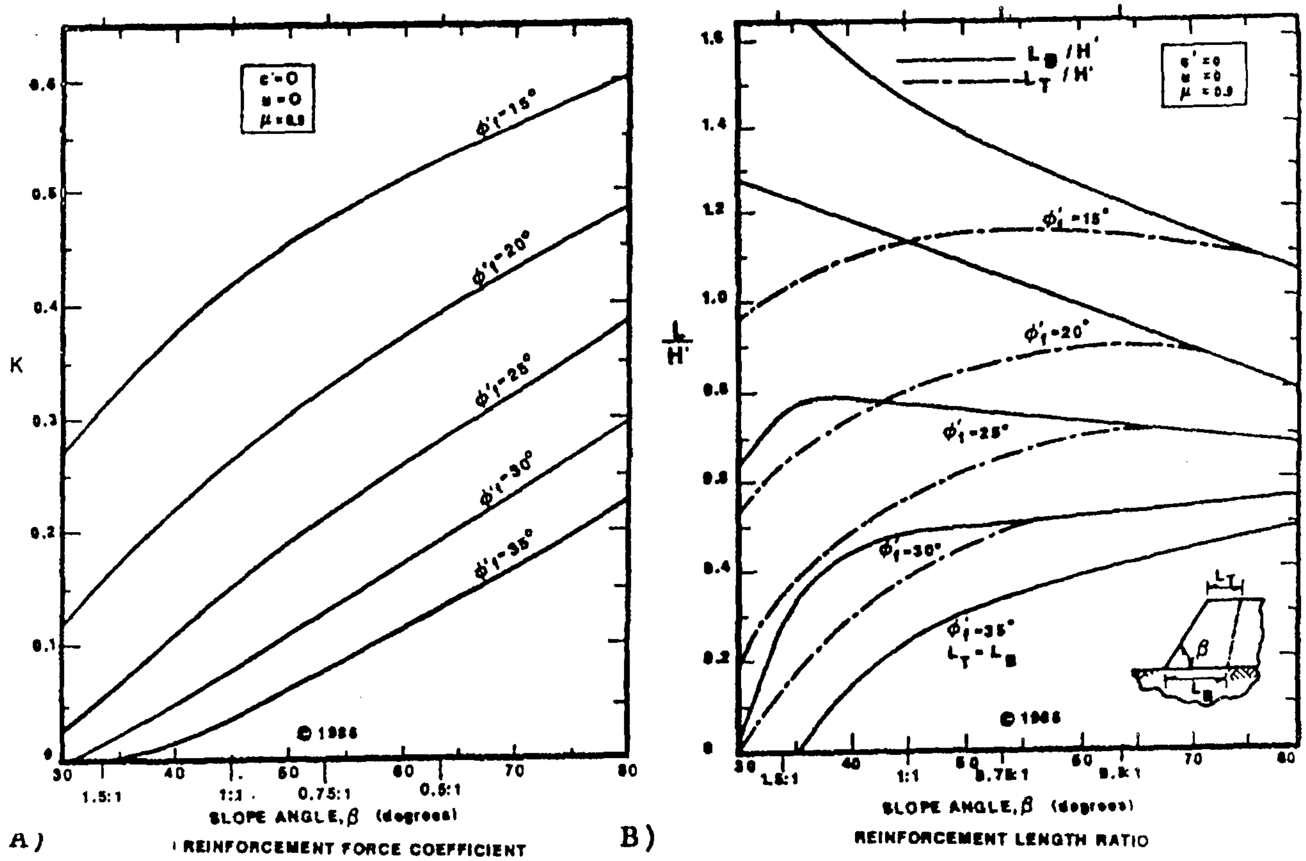


CHART PROCEDURE:

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:

$$T_{Design} = 0.5\,K\,\gamma_r\,H'^2$$

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).

For 2 zones:

$$T_{Bottom} = 3/4\ T_{s\text{-}max} \tag{11}$$

$$T_{Top} = 1/4\ T_{s\text{-}max} \tag{12}$$

For 3 zones:

$$T_{Bottom} = 1/2\ T_{s\text{-}max} \tag{13}$$

$$T_{Middle} = 1/3\ T_{s\text{-}max} \tag{14}$$

$$T_{Top} = 1/6\ T_{s\text{-}max}$$

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

Determine reinforcement vertical spacing S_v or the design tension T_d requirements for each reinforcement layer.

- For each zone, calculate T_d 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:

$$T_d = T_a R_c = \frac{T_{zone} S_v}{H_{zone}} = \frac{T_{zone}}{N} \tag{15}$$

where:

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

S_v = Use 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_{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.

Maximum vertical spacing of 600 mm or less should be designed to ensure face stability or compaction quality.

For detailed analyses required for final design, refer to FHWA-SA-96-071, *Mechanically Stabilized Earth Walls and Reinforced Soil Slopes, Design and Construction Guidelines.*

c. Facing Options

Slope facing requirements will depend on soil type, slope angle and reinforcement spacing. Table 6 may be used as a guide in selection.

Table 6. 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	Hard Facing	Vegetated Face	Hard Facing
> 50° All Soil Types	Not Recommended	Gabions	Sod Permanent Erosion Blanket w/ seed	Wire Baskets Stone Shotcrete
35° to 50° Clean Sands (SP) Rounded Gravel (GP)	Not Recommended	Gabions Soil-Cement	Sod Permanent Erosion Blanket w/ seed	Wire Baskets Stone Shotcrete
35° to 50° Silts (ML) Sandy Silts (ML)	Bioreinforcement Drainage Composites	Gabions Soil-Cement Stone Veneer	Sod Permanent Erosion Blanket w/ seed	Wire Baskets Stone Shotcrete
35° to 50° 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° to 35° 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

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 addressing 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 explicitly 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 developed 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 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 apply 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.
- 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 introduction by numerous suppliers of a variety of complete systems or components that are applicable for use. Alternately, 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, offered subsequently.

For the purpose of prior approval, it is desirable that the supplier submit a data 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, and with illustrations as necessary, the step-by-step construction sequence.
- 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 bench-mark for comparison with systems that have been in successful use and 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 1998 Interim AASHTO Section 5.8 *Mechanically Stabilized Earth (MSE) Walls*, or methods outlined in FHWA-SA-96-071, Mechanically Stabilized Earth Walls and Reinforced Steepened Slopes, Design and Construction Guidelines for both MSE 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, material and construction 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 should 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.

- Elevation views indicating elevations at top and bottom of walls or slopes. Beginning and end stations, horizontal and vertical break points, 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 layout and the designation of the type or module, the elevation of the top of levelling pad and footings, the distance along the face of the wall to all steps in the footings and levelling 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.

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 drainage schemes if required.

f. Specifications

Construction and material specifications for the applicable system or component as detailed later in this chapter.

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 preapproved 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.

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.

These 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.

- 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.

- 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.

5.6 REVIEW AND APPROVALS

Where agency design is based on suppliers plans, it should be approved for incorporation in 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-SA-96-071, *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:

- Levelling 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

CASE HISTORIES AND COST DATA

6.1 MSE CONSTRUCTION

The following case histories are presented to provide representative examples of cost-effective, successful MSE applications.

a. I-94 wall using MBW facing units

The first segmental block-faced MSE wall specified by Mn DOT 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 long and has a maximum height of 4.3 m. The completed structure is shown on figure 19.

During conceptual design, a low gravity wall 1 to 1.5 m was envisioned at this location with retention provided by MBW units, without soil reinforcement elements. During final design, heights up to 3.2 m became necessary, and the contract documents therefore included for a contractors design for a 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.

Design

The project specifications required that the contractor provide a design in accordance with the manufacturers 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 Mn DOT requirements for granular borrow. These requirements specify a maximum size of 25 mm and no more than 20 percent 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 percent of the height. The minimum vertical spacing was 200 mm at the bottom of the wall with 600 mm spacing at the top.

Figure 19. 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.

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. Horizonal 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 batter from the trailing lip. The top of the wall at the tallest section of 4.3 m was therefore 711 mm 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 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 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 section of wall which bowed outward when the wall was about 3 m high, was observed on the first section of wall built. The bow was eliminated by removing the facing block, drain soil, 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 to be governed by all applicable requirements of AASHTO, Section 5.8 as amended by Mn DOT.
- Wall systems to be used on future project must have pre-approved components by Mn DOT and that the contractor identify his chosen system in the bid.
- Standards for the manufacture of MBW blocks with respect to compressive strength and water absorption be developed by Mn DOT.
- Erection tolerances on the order of 25 mm per 3 m of height must be included in the specifications.

Cost

The bid cost was $270 per square meter installed. Cast-in-place concete walls on the same project were bid at $409 per square meter.

b. <u>US 23 (I-18 Extension) Unicoi Co. Tenn, using precast concrete facing units</u>

The use of superimposed MSE structures with precast concrete facings for a combined height in excess of 30 m 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 N.C. 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.

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 a processed on site gneiss rock, from cut areas, crushed, and screened to a maximum size of 75 mm.

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.

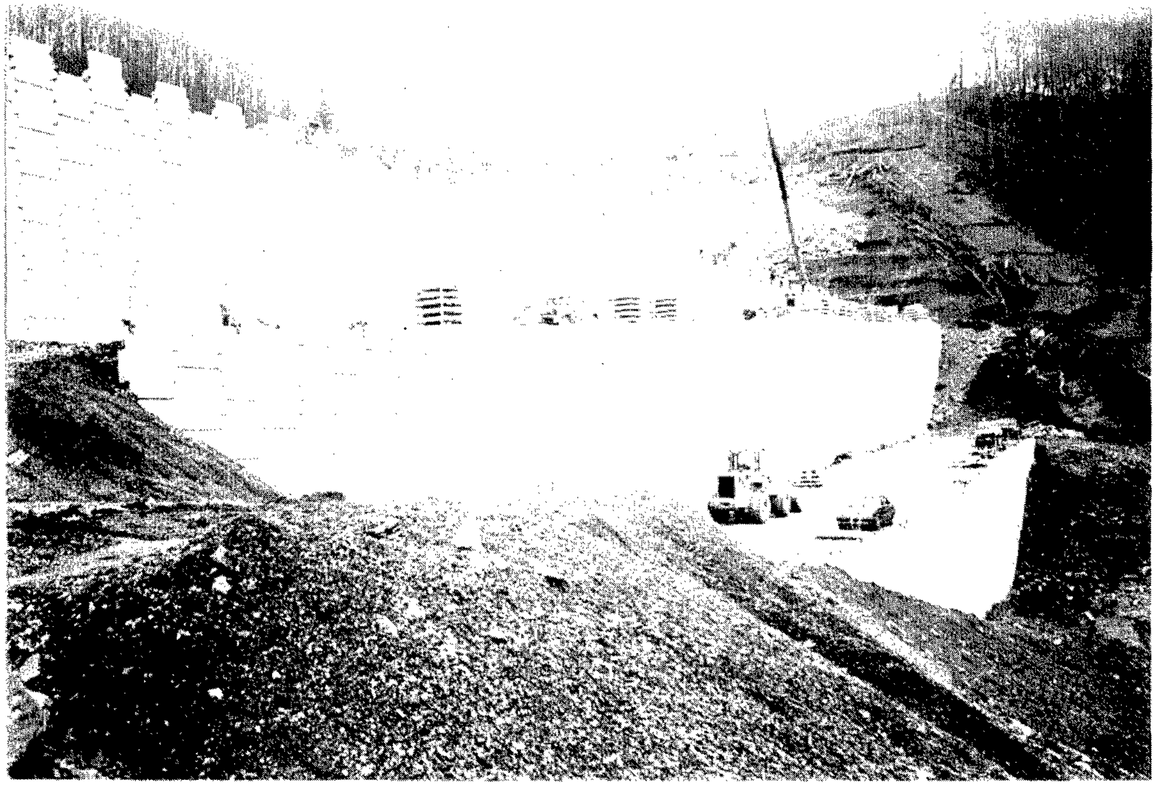
The submitted designs were checked for compliance to the geometric and design requirements and the unique pay quantities included in the bid summary sheets. The successful bidder was required to specify the system chosen to preclude post bid shopping by the general contractor. The low bid price was $313/m^2$, which included the placement and compaction of the select fill within the reinforced soil zone. The completed construction at a tiered location shown on figure 20, proceeded with no difficulty at an average erection rate of 60 square meters per day.

6.2 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

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.



a) During erection

b) Completed

Figure 20. US 23 superimposed MSE walls.

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 to 18 m 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 to 18.3 m long and spaced 0.6 to 1.2 m vertically throughout the reinforced embankment. The ultimate strength of the primary reinforcement was on the order of 100 kN/m. 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 with a vertical spacing of 0.3 m 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 and has been periodically monitored by visual inspection and slope inclinometers. Project photos are shown in figure 21. 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.

b. Salmon-Lost Trail Roadway Widening Project

As part of a highway widening project in Idaho, the Federal Highway Administration designed and supervised the construction of a 172-m-long, 15.3-m-high, 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. Aesthetics 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

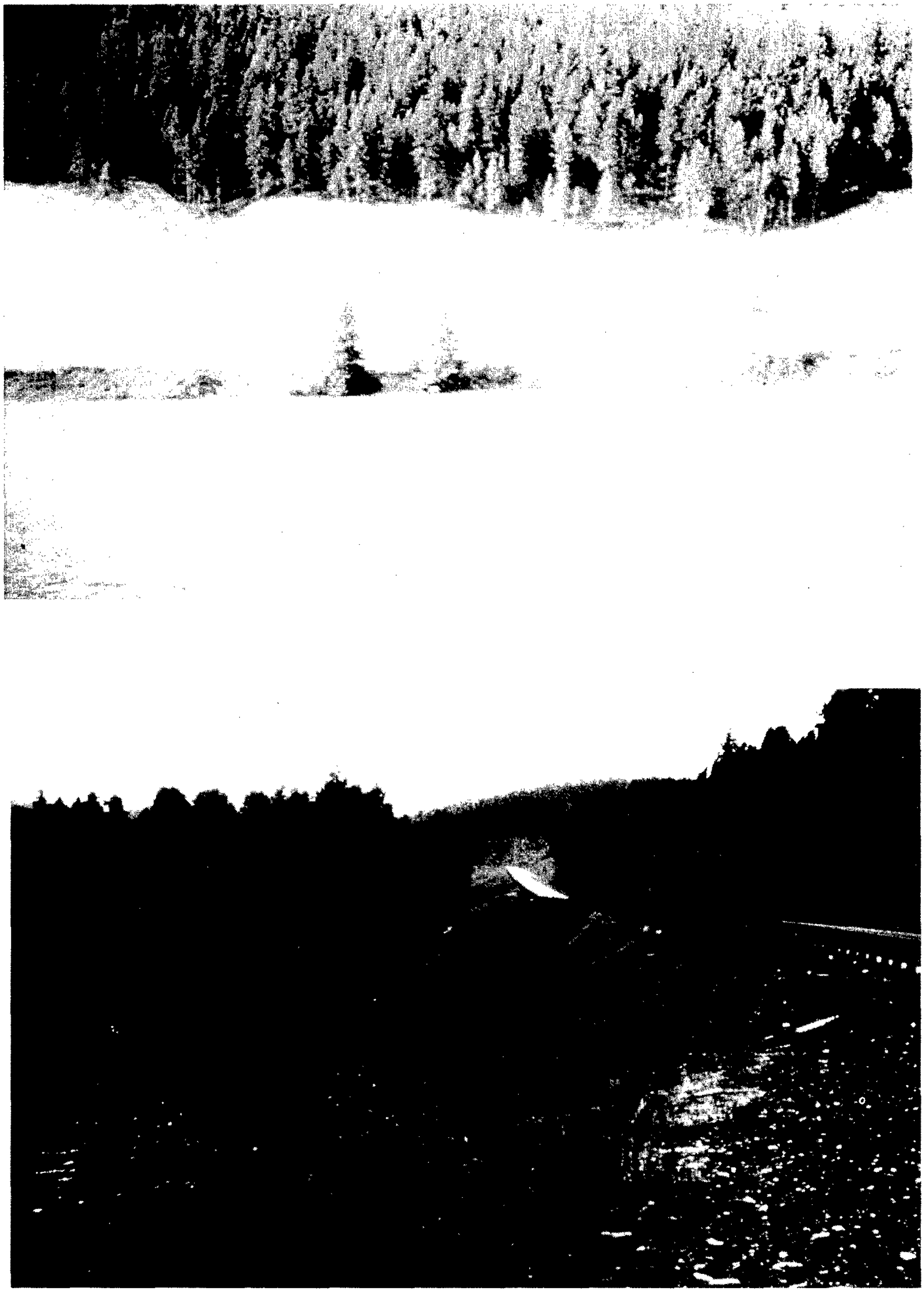


Figure 21. Dickey Lake Site.

seeps or weeps coming out of the existing slope. Geotextile reinforcements with an inplane 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. The reinforcement in the lower zone had an ultimate tensile strength of 100 kN/m, and the reinforcement in the upper zone had a reinforcement strength of 20 kN/m. 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 percent reduction in reinforcement).

The construction was completed in 1993 (see figure 22 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 to 0.2 percent of the height of the slope, with maximum strains in the reinforcement measured at only 0.2 percent. 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 $\$160/m^2$ of vertical face.

6.3 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, and 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 to 4.5 m and average foundation conditions. Modular block walls (MBW) are competitive with concrete walls at heights of less than 4.5 m.

Figure 22. Salmon Lost Trial Site.

In general, the use of MSE walls results in savings on the order of 25 to 50 percent and possibly more in comparison with a conventional reinforced concrete retaining structure, especially when the latter is supported on a deep foundation system. A substantial savings is obtained by elimination of the deep foundations, which is usually possible because reinforced soil structures can absorb relatively large total and differential settlements. Other cost saving features include ease of construction and speed of construction. Typical total costs range from $160 to $300 per m^2 of face, generally as function of height and cost of select fill.

The actual cost of a specific MSE 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 to 30 percent of total cost.
- Reinforcing materials - 20 to 30 percent of total cost.
- Facing system - 25 to 30 percent of total cost.
- Backfill materials including placement - 35 to 40 percent of total cost, where the fill is a select granular fill from an off-site borrow source.

In addition, consideration must be given to the cost of excavation, which may be somewhat greater than for other systems.

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 are often necessary for steeper embankment slopes and additional costs such as slope protection or facings 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 and 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; and aesthetics.

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 to 65 percent of total cost.
- Backfill - 30 to 45 percent of total cost.
- Face Treatment - 5 to 10 percent of total cost.

High RSS structures have relatively higher reinforcement and lower backfill costs. Recent bid prices suggest costs ranging from \$110 m^2 to \$260 m^2 as a function of height.

For applications in the 10-to-15 m height range, bid costs of about \$170 m^2 have been reported.

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:

- *FHWA Tour for Geotechnology - Soil Nailing,* May, 1993.
- *Soil Nailing Field Inspectors Manual,* FHWA SA-93-068, 1994.
- *Recommendations Clouterre,* FHWA SA-93-026, 1991 (English Translation).
- *Soil Nailing for Stabilization of Highway Slopes and Excavations,* FHWA RD 89-198, 1989.
- *Manual for Design and Construction Monitoring of Soil Nail Walls,* FHWA-SA-96-069, 1996.

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 is 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 on 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 1970's 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 20 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 on-going research to formulate common design methods, construction techniques, and material specifications.

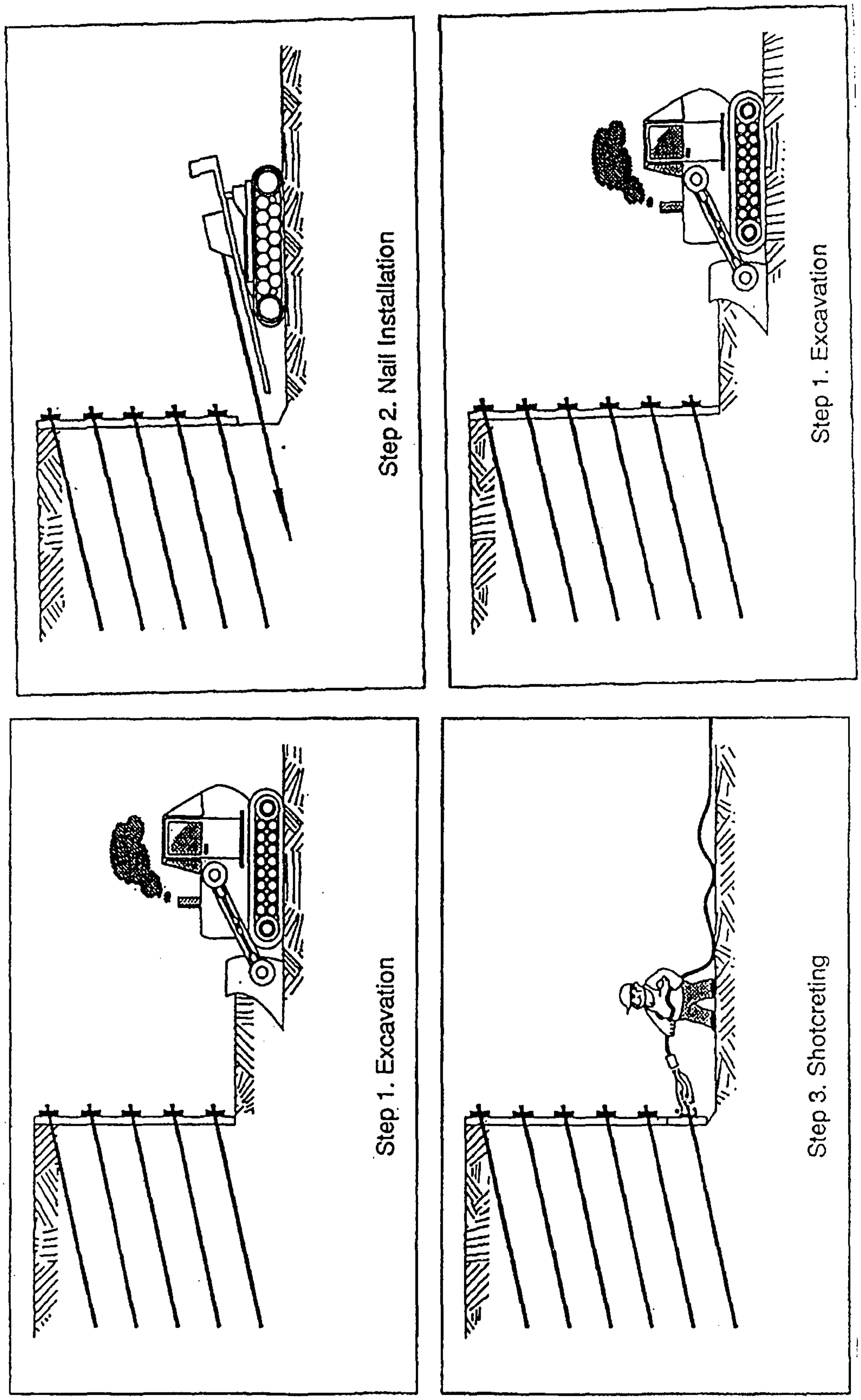

Figure 1. Soil Nail Wall Construction Sequence.

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
- Design and construction concepts
- 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-SA-96-069 "Manual for Design and Construction Monitoring of Soil Nailed Walls."

For current in depth recommendations on field inspection and construction methods, the reader is 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 most 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 to 30 percent. A wide range of aesthetic requirements can be accommodated by casting concrete in place over the shotcrete facing or by texturing the shotcrete or by the use of 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 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 bridge. 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.

- 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:

- Incorporation of a temporary excavation system into the permanent facility.

- Reduction in cut excavation and rock blasting.

- Reduction in concrete quantities used.

- Elimination of deep foundations for support.

- Potential reduction of right-of-way acquisition and less environmental impact.

- Reported lower cost 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, as would be the case for a tieback 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 shares with other cut retaining techniques the following disadvantages:

- Permanent underground easements may be required.

- Groundwater 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 prestressed 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.

- 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 based by considering a number of factors namely:

- Geometric constraints.
- Suitability of site soil and groundwater conditions.
- Displacements.
- Cost considerations.

The effects of these factors are discussed below.

Geometric Constraints

The required length of nails for construction can vary between 60 and 120 percent of the wall height, but more typically from 70 to 90 percent 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 to 1 meter 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 in the following soil profiles:

- Soils containing organic matter and characterized by low strengths.
- Cinder, ash or slag fills.
- Rubble fills or industrial wastes.
- Acid mine wastes.

- Cohesive soils with liquid limit (LL) greater than 50 and with plasticity index (PI) greater than 20 must be carefully assessed for creep susceptibility. Experience from tiebacks suggests that soils with consistency index (I_c) of less than 0.9 are susceptible to creep. These soils should not be considered for permanent soil nailed structures without extensive investigations of their creep potential and subsequently designed for working loads well below the critical creep load.

The consistency index I_c is defined as:

$$I_c = \frac{W_1 - W}{W_1 - W_p}$$

where: I_c=Consistency Index
W_1=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 a temporary face stabilization berm.

- Soil Strength - In cohesionless soils, current practice precludes the use of ground anchors when the soils are in a loose state (SPT-N < 5). 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. Similar limitations for nailed structures are warranted at present.

- Below permanent ground water 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 on table 1.

Table 1. Estimated structure deflection.

	Weathered Rock	Sand	Clays
Vertical or horizontal deflection at top of wall face	H/1000	2H/1000	4H/1000

Designs based on lower global factors of safety (FS < 1.5) or with steep nail inclinations (>20 degrees) may develop deflections greater than the range indicated on table 1.

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 or tieback wall construction.

For temporary construction as part of a feasibility assessment, a cost range of $170/m^2 to $400/m^2 can be assumed. For permanent construction with a precast or cast-in-place concrete spacing, a cost range of $400/m^2 to $600/m^2 can be assumed. In general, these costs are on the order of 15 percent 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 to 2 m for one to two days, to allow nail and temporary shotcrete construction.

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 to 5 kPa associated with a moisture content of at least 5 to 6 percent (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 uncemented 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 generally proceeds as follows:

1. An initial 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. Immediately after excavation, the freshly exposed surface is covered with a reinforced shotcrete layer.

4. 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. Alternately, soil nails may be driven.

5. Steps 3 and 4 may be inverted.

6. The nails may be prestressed to a small percentage of their working loads, against a small plate secured on the initial shotcrete layer. The prestress load usually does not exceed 20 percent 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.

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 overexcavate. 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

Figure 2. Excavation of first lift and placement of drainage strips.

Figure 3. Drilling a nail hole.

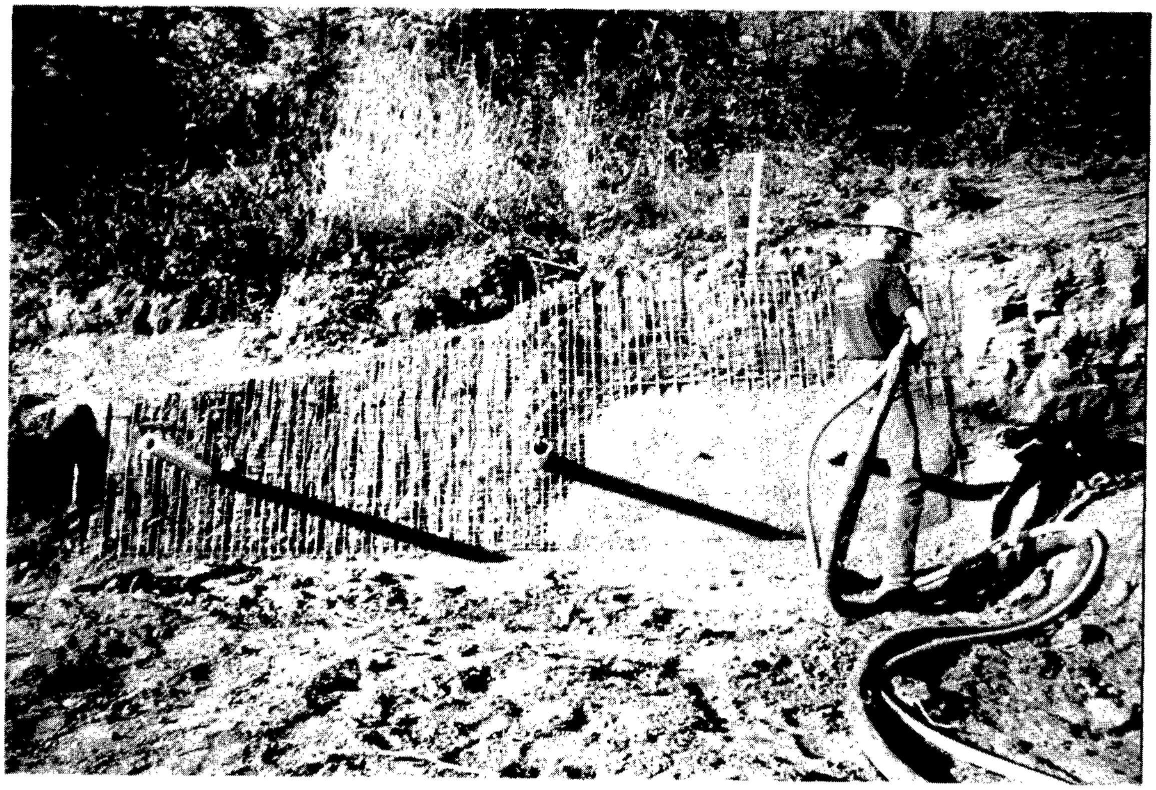

Figure 4. Shotcreting of face.

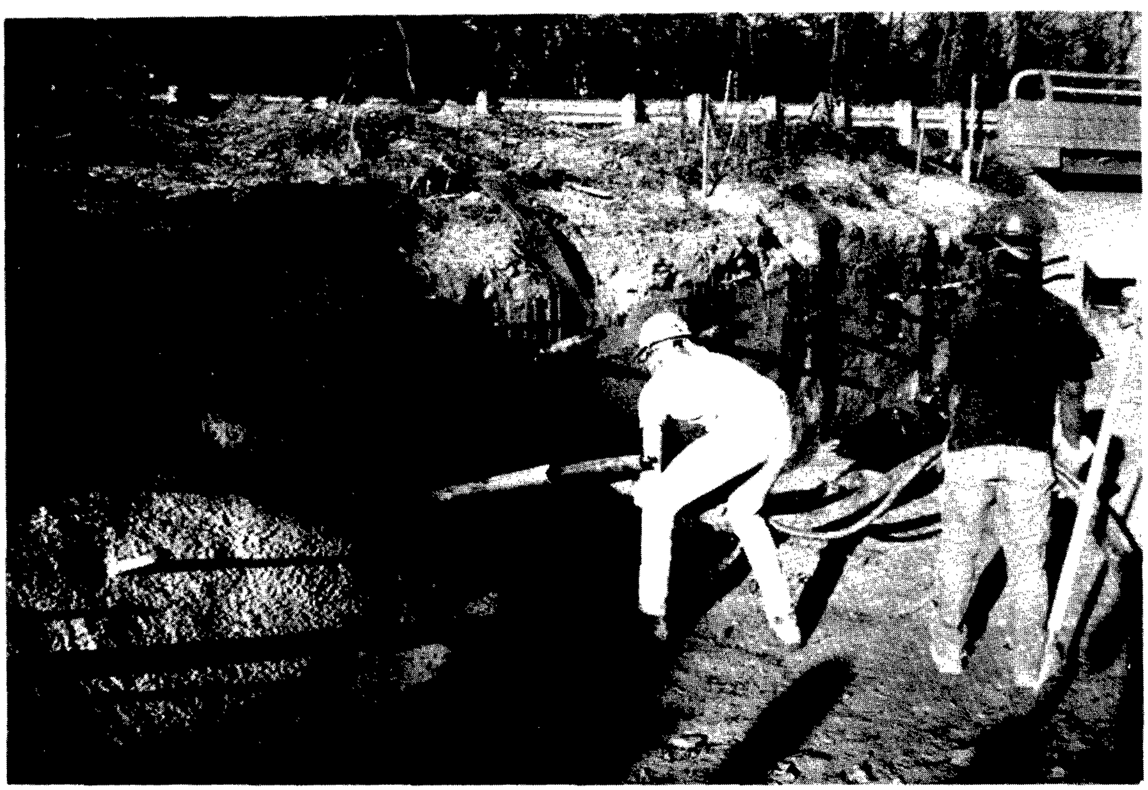

Figure 5. Grouting a nail hole.

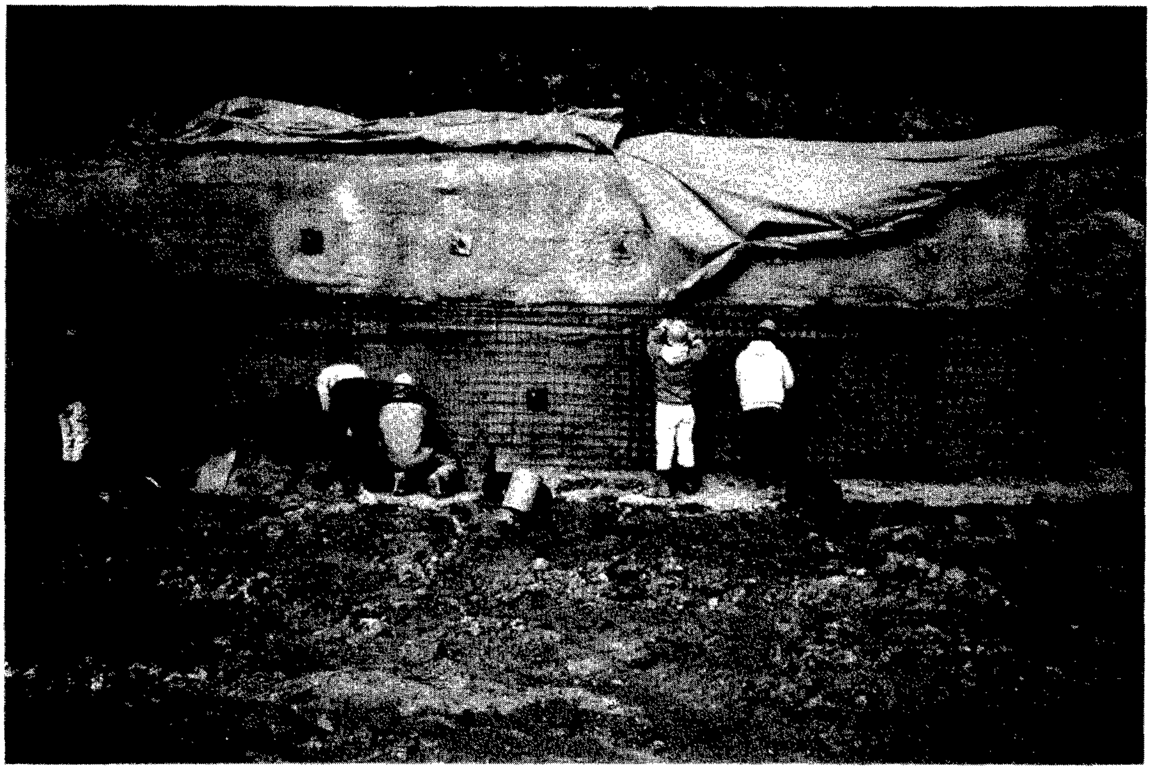

Figure 6. Steel placement prior to second stage Shotcrete layer at facing.

Figure 7. Construction of cast-in-place final facing.

- Driven nails

- Jet grouted nails

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 either 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 is typically wider, from 1.2 to 1.8 m on center. Drill-hole diameter will vary from 100 mm with rotary drilling to 300 mm 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 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 include percussive methods which displace soil by driving drill rods 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 a diameter of 90 mm to 115 mm.

Cased hole methods of drilling are also used to install soil nails. This more expensive drilling method 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 mm to 140 mm 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 creating a homogenous composite reinforced soil mass.

The nails are driven using vibropercussive 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 but to date not in the United States. A similar proprietary technique using explosive for driving of steel nails using an air gun was developed in the United Kingdom and recently demonstrated in the United States. It appears to be particularly well suited for small slope stabilization projects.

Jet grouted nails are of a composite construction made from a grouted soil with a central steel nail installed simultaneously. They are used for temporary applications and may be used for permanent applications if the degree of required corrosion protection is low. Nails can be installed using vibropercussion driving at high frequencies (up to 70 Hz) and extremely high grouting pressures (13,000 KPa).[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 the United States to date.

Alternately, significantly lower grouting pressures (1,400 KPa) have been used in practice with a variety of nails including hollow bars (TITAN, Chance IBO, etc.) which are used as the drill stem initially and then disconnected and left in the hole to serve as the structural members. This technique has been used in the United States, primarily on private commercial projects.

Jet grouting techniques provide in addition, recompaction and improvement of the surrounding soil and can increase, significantly in granular soils, the shear and pullout resistance of the soil.

c. Grouting

For open hole drilling, low pressure tremie grouting is normally used. To increase capacity, regrouting 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 to 0.5 by weight, achieving a compressive strength of 21,000 KPa. Regrouting is not commonly used because it increases costs, significantly. Grouting can be performed prior to nail installation or after.

3.3 MATERIALS OF CONSTRUCTION

a. Nails

Ductile steel bars with a yield strength in the range of 420 to 520 N/mm^2, with or without corrosion protection, are typically used for conventional drill and grout soil nails. Typical diameters range from 19 mm to 38 mm. 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 or during grouting 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 spacing (3 m 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 to 100 mm 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 and 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 have 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.(3)

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 tolerances are obtained by vertical connecting slots in assemblies connected at the nail head. 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 the back of the precast panels. The space between them is then filled with the low strength concrete to form the connection.

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.

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 failure
- External failure

Schematically, these modes are illustrated on figure 8.

The basic concept underlying the design of soil nailed structures relies on:

- 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.
- Passive resistance developed 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. Nails placed across a potential slip surface can resist the shear and bending moment through the development of passive resistance. 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.

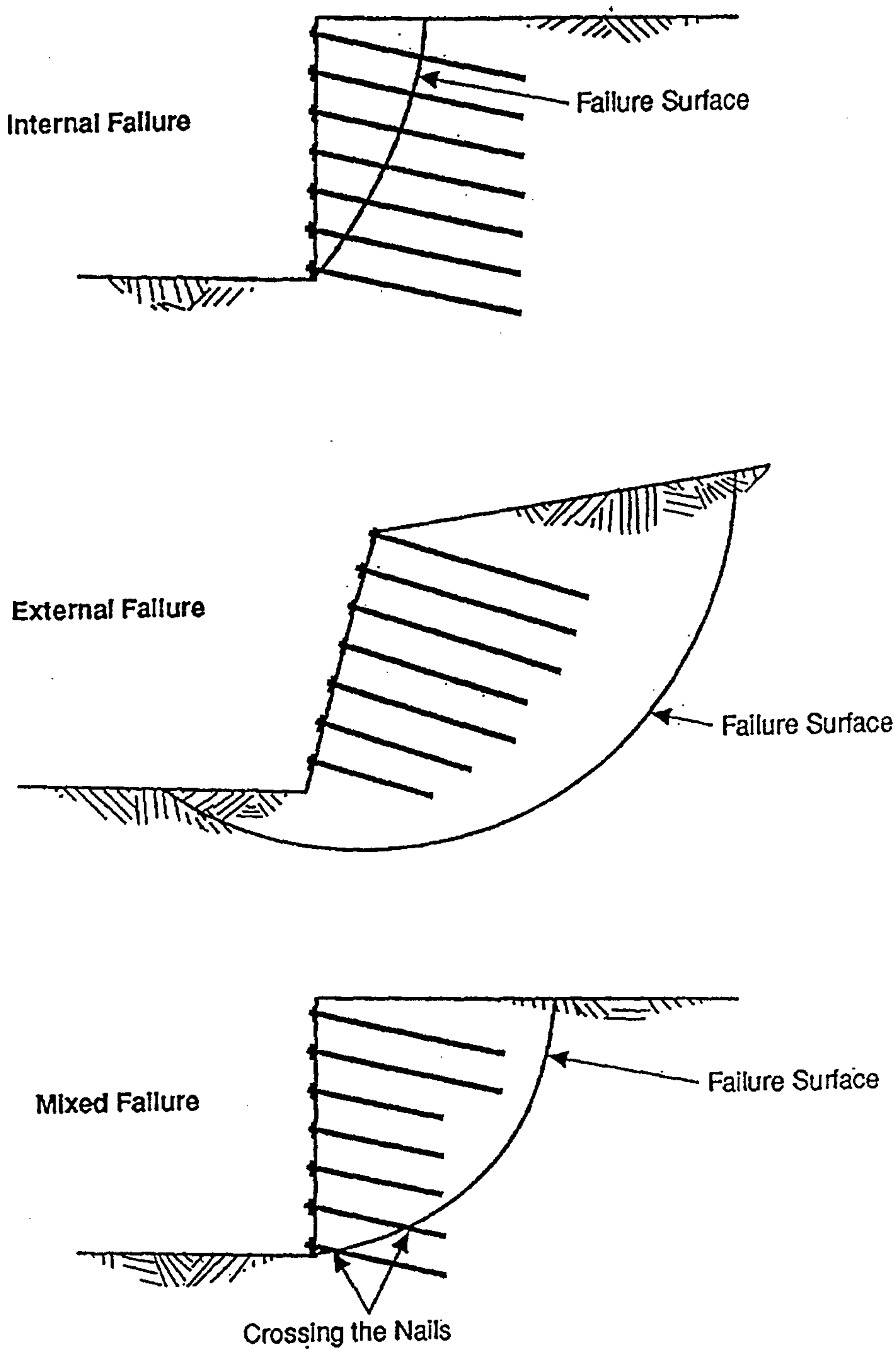

Figure 8.
Different types of failure surfaces to be analyzed for soil nailed walls.

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 on figure 9.

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. They make no assumption on how each of the installed nails contribute to the overall required stabilizing force and do not consider the influence or effect of the facing.

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 consider in addition the effects of shear capacity and bending stiffness of the nails on the overall structure stiffness.

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 on table 3 with respect to essential features.

An improvement to the limit equilibrium methods that considers 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 has been developed. The details are outlined in FHWA-SA-96-069.

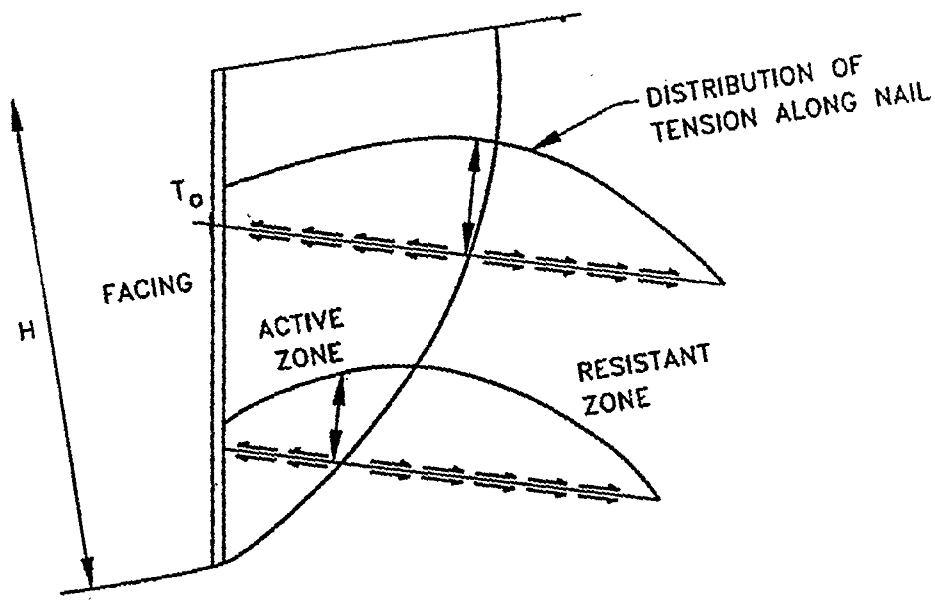

Figure 9. Conceptual soil nail behavior.

Table 3. Soil nail design methods.

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, proprietary
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
6. Golder Assoc. GOLDNAIL (1991)	Limit Force Equilibrium, Global Stability	Circular	Pull-out, nail yield stress	Global F.S. Critical Failure Surface, Considers influence of facing	U.S.	GOLDNAIL, proprietary

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.

Because of the variables involved, there is wide spread agreement 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 frictional resistance for grouted nails, F_ℓ values based on data from FHWA RD 89-198, FHWA-SA-96-069 and pressuremeter correlations in Europe should be considered for preliminary design evaluation. They are summarized on tables 4 through 6.

Table 4. Estimated pullout capacity in rock.

Construction Method	Rock Type	Ultimate Pullout Strength F_ℓ (KPa)
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

Table 5. Estimated pullout capacity in cohesionless soils.

Construction Method	Soil Type	Ultimate Pullout Strength F_ℓ (KPa)
Rotary Drilled	Silty sand Silt Piedmont residual Fine colluvium Sand/gravel	100 - 150 60 - 75 40 - 120 75 - 150 100 - 180
Driven Casing	Sand/gravel (low overb.) Sand/gravel (high overb.) Dense Moraine Colluvium	190 - 240 280 - 430 380 - 480 100 - 180
Jet Grouted	Sand Sand/gravel	380 700
Augered	Silty sand fill Silty fine sand Silty clayey sand	20 - 40 55 - 90 60 - 140

Table 6. Estimated pullout capacity in cohesive soils.

Construction Method	Soil Type	Ultimate Pullout Strength F_ℓ (KPa)
Augered	Loess Soft clay Stiff clay Clayey silt Calcareous sandy clay	25 - 75 20 - 30 40 - 60 40 - 100 90 - 140
Driven Casing	Clayey silt	90 - 140
Rotary Drilled	Silty clay	35 - 50

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 fully outlined in FHWA RD 89-198 and in FHWA-SA-96-069.

a. *Factors of Safety*

Designs based on limit equilibrium factors of safety calculate a global 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 use a factor of 1.35 to 1.5 for "critical structures." To date, Service Load Design (SLD) methods have been used exclusively.

Alternatively, partial factors of safety may be assigned to each input parameter consistent with the degree of uncertainty associated with each parameter. Under this methodology, each input parameter is factored by a unique factor of safety and limit equilibrium analyses performed, requiring a much smaller global factor of safety reflecting only the uncertainty of the calculation method used.

The following partial factors of safety have been used with this procedure:

F_G	=	Global or method F.S.	=	1.1 to 1.15
F_ϕ	=	F.S. for frictional strength	=	1.35
F_c	=	F.S. for cohesive strength	=	1.35
F_p	=	F.S. for pullout capacity	=	2.0
F_y	=	F.S. on yield strength of nail	=	1.82

In FHWA-SA-96-069, a minimum global factor of safety of 1.35 is recommended, consistent with the design procedures developed and a factors of safety for pullout capacity of 2.0 and application of normal AASHTO criteria for yield of the nail.

The application of either methodology has resulted in safe construction to date.

b. ***Preliminary Design for Feasibility Evaluations***

For preliminary design and feasibility evaluations to determine nail lengths and sizes, with simple geometrics 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 developed under FHWA-SA-96-069 are included in this Section as an example. These charts have been prepared for a common nail inclination of 15 degrees. The charts shown as figures 10 through 18 are presented in dimensionless format with the following variables:

Backslope Angle, δ

Three sets of design charts are presented (three charts per set), with each set of charts corresponding to a single backslope angle of 0, 10, or 20 degrees. For intermediate backslope angles, interpolate between the charts.

Face or Batter Angle, β

For each backslope angle, design information is presented for two face or batter angles of 0 and 10 degrees from the vertical. For intermediate face or batter angles, interpolate between the charts.

Strength Variables

Factored Friction Angle, ϕ_D

The factored friction angle of the soil is defined by the following relationship:

$$\phi_D = \tan^{-1}[\tan(\phi)/F_\phi]$$

The factored friction angle is shown on the horizontal axis of Chart A of each chart set.

<u>Dimensionless Cohesion, c_D</u>

c_D is the soil cohesion normalized with respect to the soil unit weight and the vertical height of the cut.

$$c_D = c/(Fc\gamma H)$$

The dimensionless cohesion is shown as a parameter for each slope geometry, for two values of 0 and 0.05. Interpolate for intermediate values of the dimensionless cohesion.

Dimensionless Nail Tensile Capacity, T_D

The dimensionless nail tensile capacity is the factored (α_N) nail yield strength normalized with respect to the soil specific weight (γ), the vertical height of the slope (H), and the nail spacings (S_V,S_H):

$$T_D = \alpha_N T_N/(\gamma H S_V S_H) \quad ; \quad \alpha_N = 0.55$$

The dimensionless nail tensile capacity is shown on the vertical axis of Chart A of each chart set.

Dimensionless Pullout Resistance, Q_D

The dimensionless pullout resistance is the factored (α_Q) ultimate pullout resistance (expressed as a force per unit length of nail), normalized with respect to the soil specific weight and the nail spacings:

$$Q_D = \alpha_Q Q_U/(\gamma S_V S_H) \quad ; \quad \alpha_Q = 0.50$$

The dimensionless pullout resistance is shown as being incorporated into the ratio (T_D/Q_D) on the horizontal axis of Chart B of each chart set.

c. *Preliminary Design Procedure*

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:

Step 1

Select the design chart set corresponding to the appropriate backslope angle. If necessary, interpolate results for intermediate backslope angles from those given in the charts.

For illustrative purposes, consider a soil nailed wall, battered (10 degree batter) 9.5 m in height with a 20° backslope angle and nail spacing of 1.5 m installed at 15 degrees below horizontal. Based on soil conditions, a unit weight of 18 kN/m^3, ϕ = 34°, c = 5 kN/m^2 and Q_U = 60 kN/m appear appropriate soil parameters. Based on recommendations in FHWA-SA-96-069, a factor of safety of 1.35 is recommended when using chart solutions.

Step 2

Compute the factored soil friction angle, ϕ_D and the dimensionless factored soil cohesion c_D as defined above. From the appropriate chart A, determine the dimensionless nail tensile capacity, T_D.

$$\phi_D = \tan^{-1}[\tan(\phi)/F_\phi] = \tan^{-1}[\tan(34^\circ)/1.35] = 26.5^\circ$$

$$c_D = c/(F_C \gamma H) = (5.0\ \text{kN/m}^2)/[1.35(18.0\ \text{kN/m}^3)(9.50\ \text{m})] = 0.022$$

From Chart A (figure 16), T_D = 0.23

Step 3

The required nail yield strength can then be determined from the relations presented and from knowledge of the dimensionless nail tensile capacity (calculated), the soil unit weight, the vertical height of the slope, the vertical and horizontal nail spacings, and the nail strength factor.

$$T_D = \alpha_N T_N/(\gamma H S_V S_H)$$

$$T_N = \gamma H S_V S_H T_D/\alpha_N = (18.0\ \text{kN/m}^3)(9.50\ \text{m})(1.50\ \text{m})(1.50\ \text{m})(0.23)/(0.55)$$

$$T_N = 161\ \text{kN (Required nominal nail strength)}$$

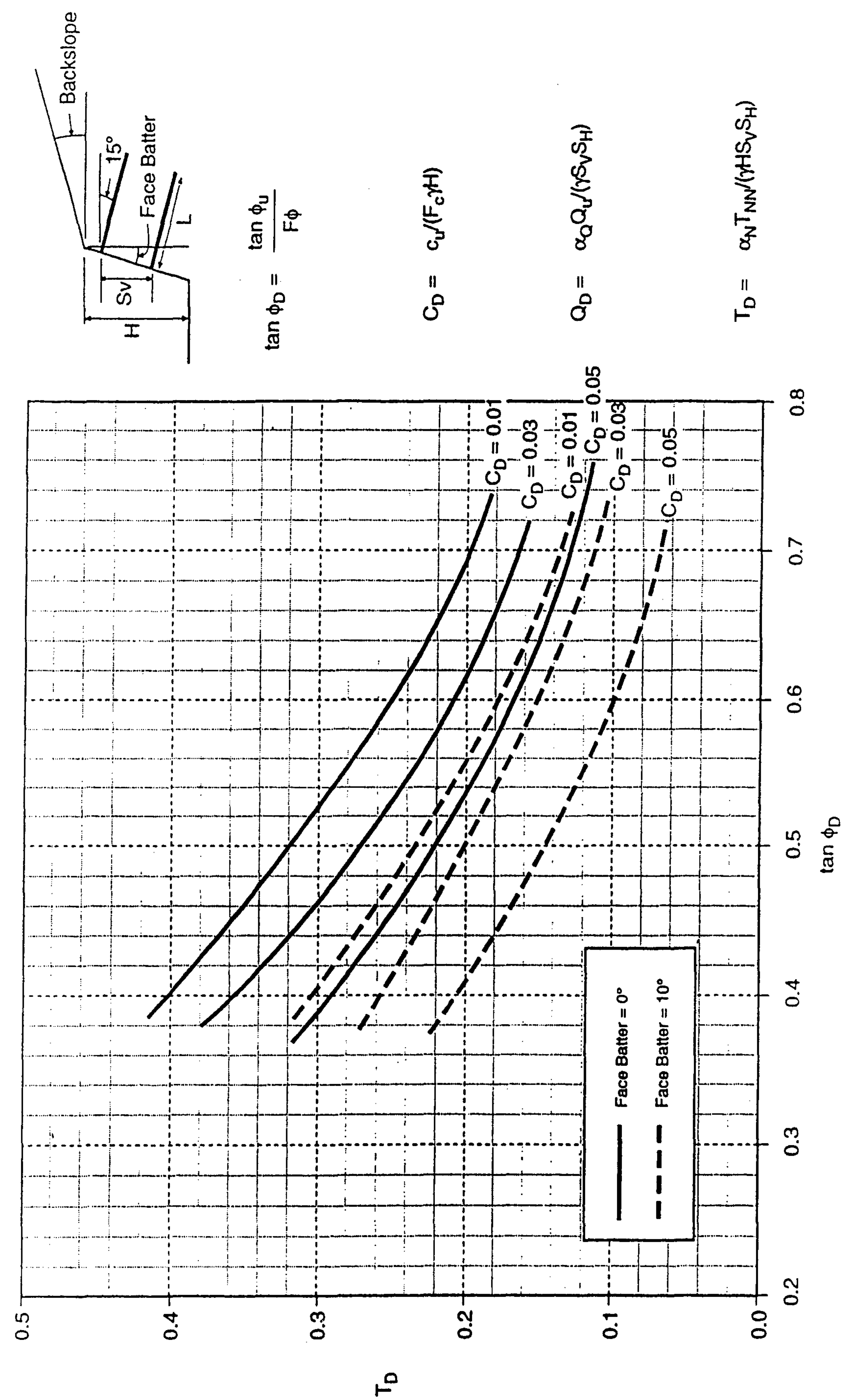

Figure 10. Design Chart Set 1: Backslope = 0° (Chart A).

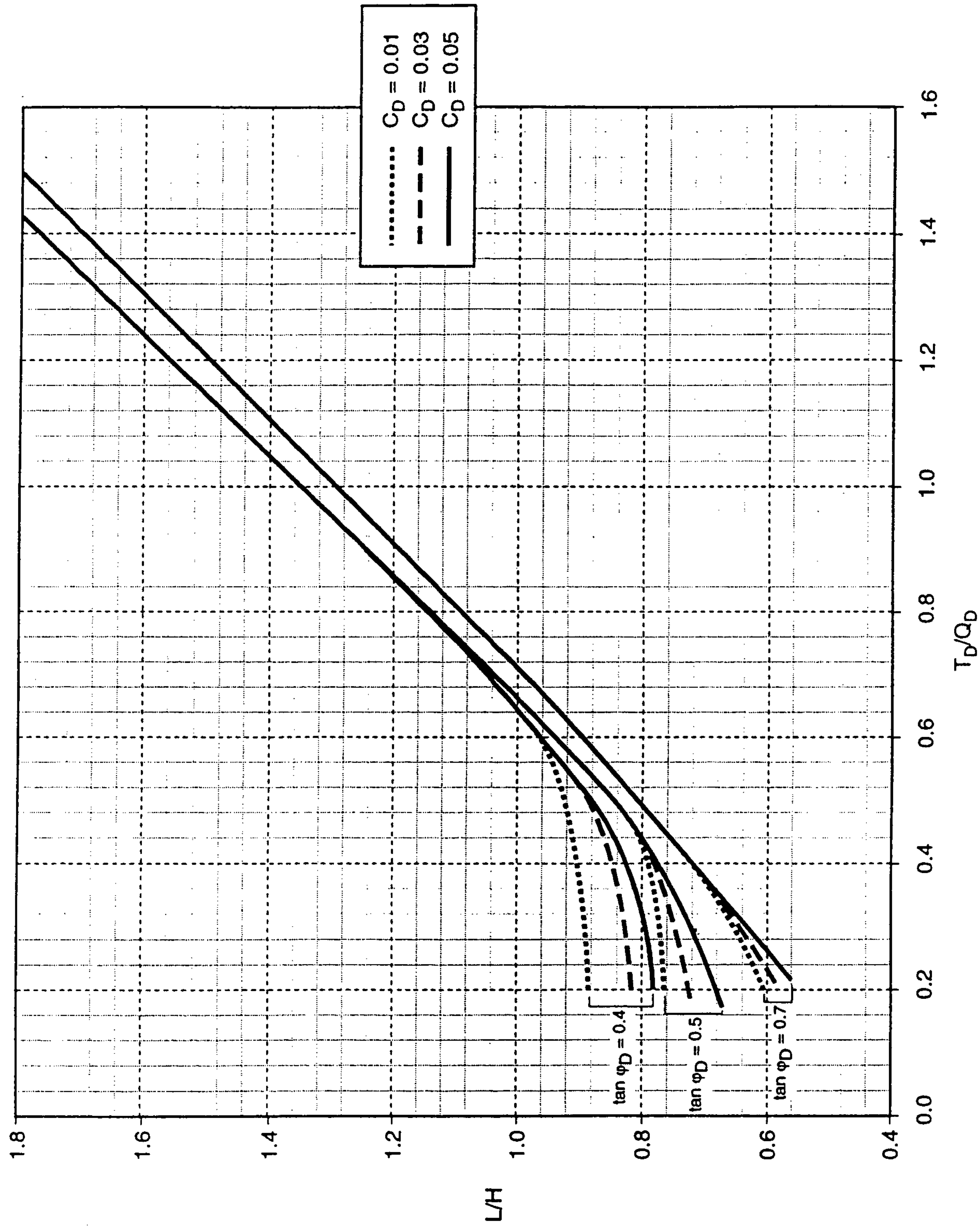

Figure 11. Design Chart Set 1: Backslope = 0°, Face Batter = 0 (Chart B).

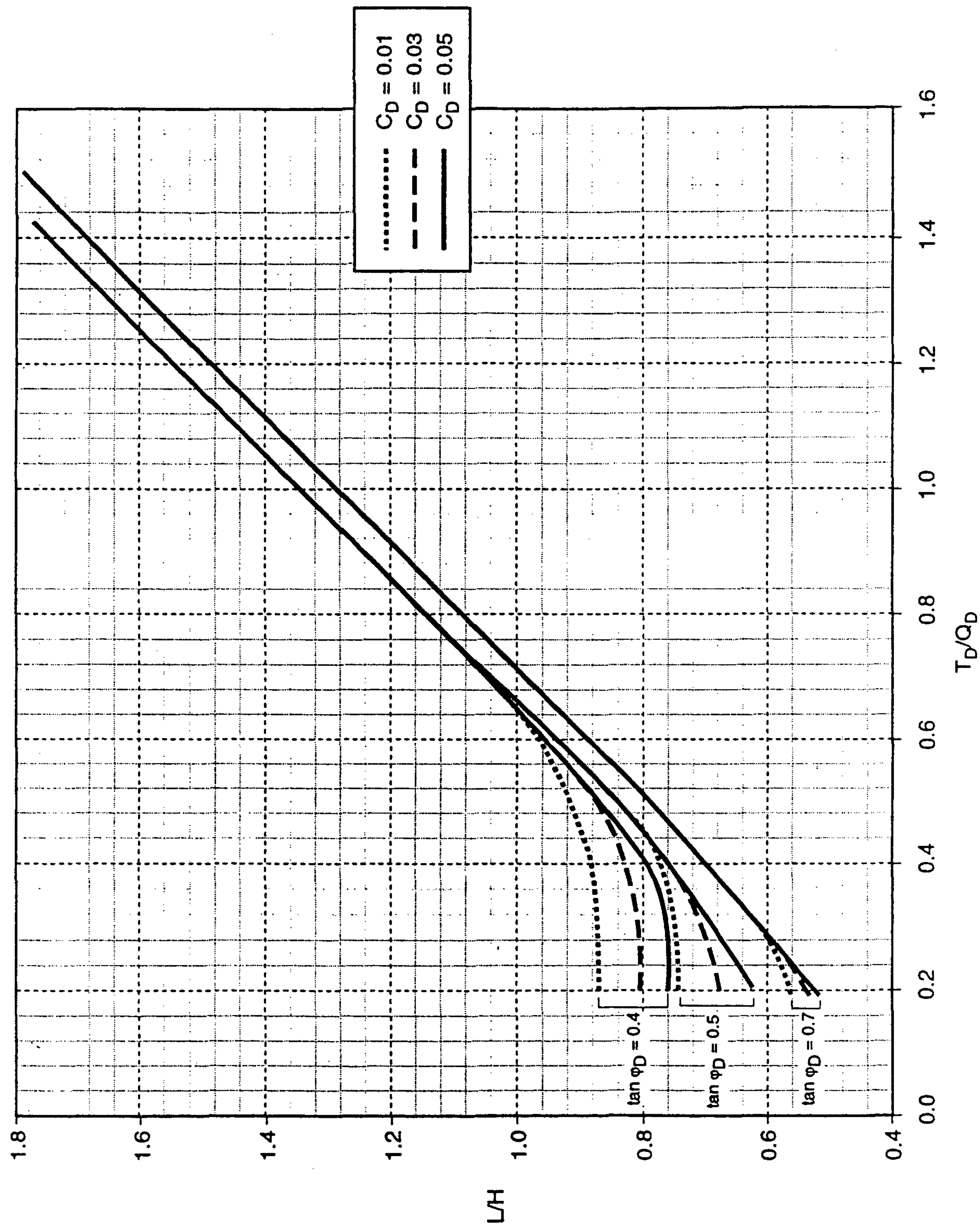

Figure 12. Design Chart Set 1: Backslope = 0, Face Batter = 10° (Chart A).

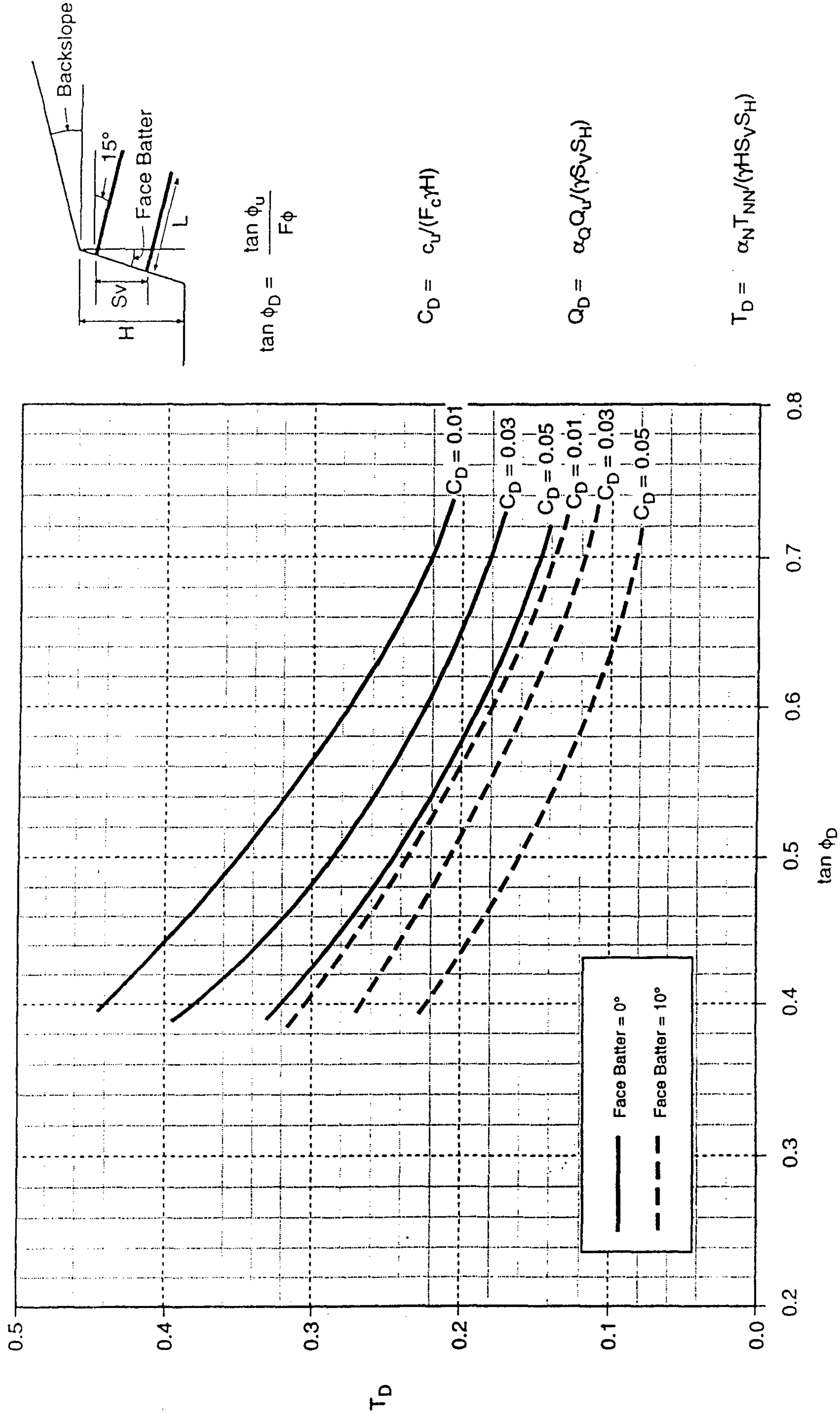

Figure 13. Design Chart Set 2: Backslope = 10° (Chart A).

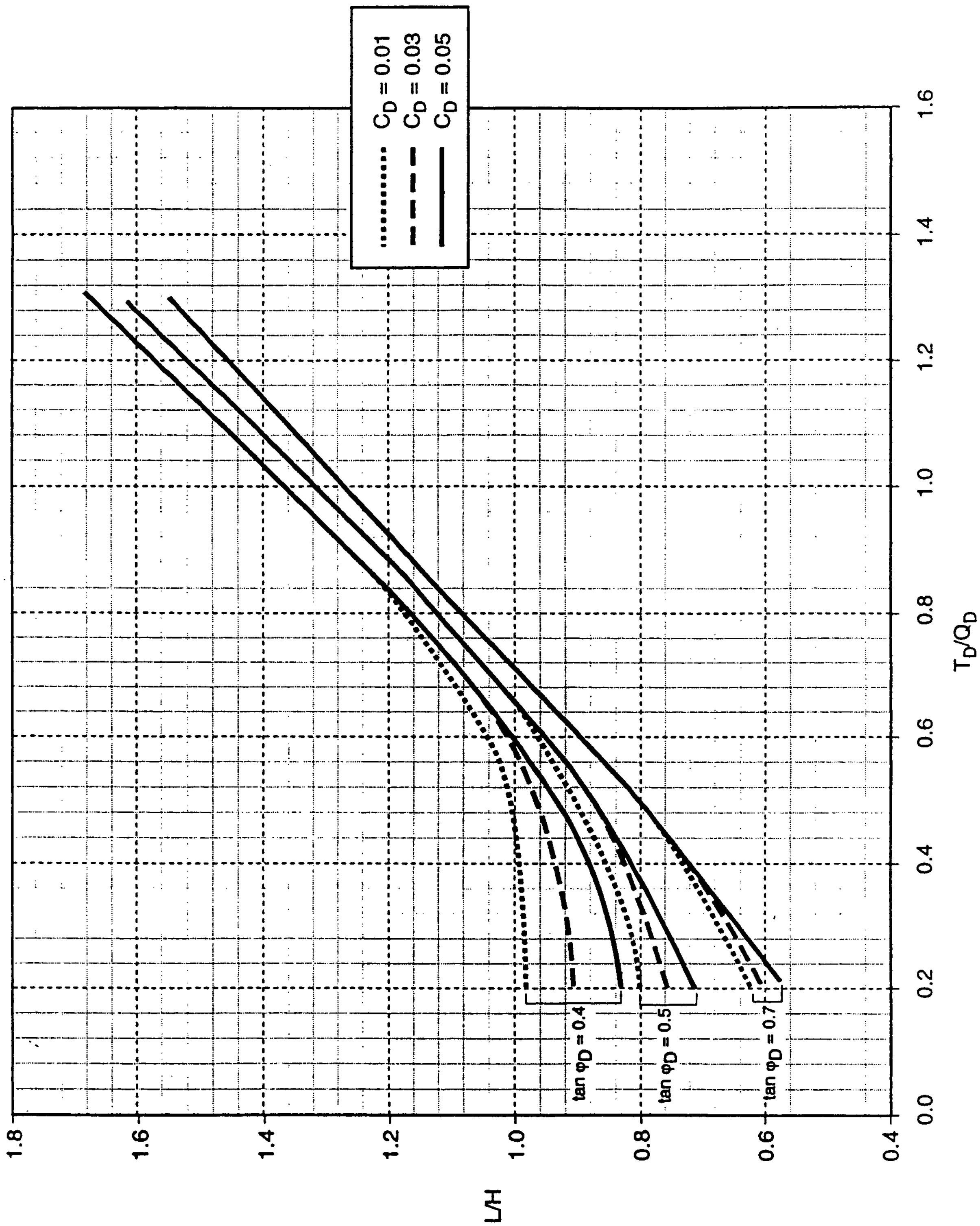

Figure 14. Design Chart Set 2: Backslope = 10, Face Batter = 0° (Chart B).

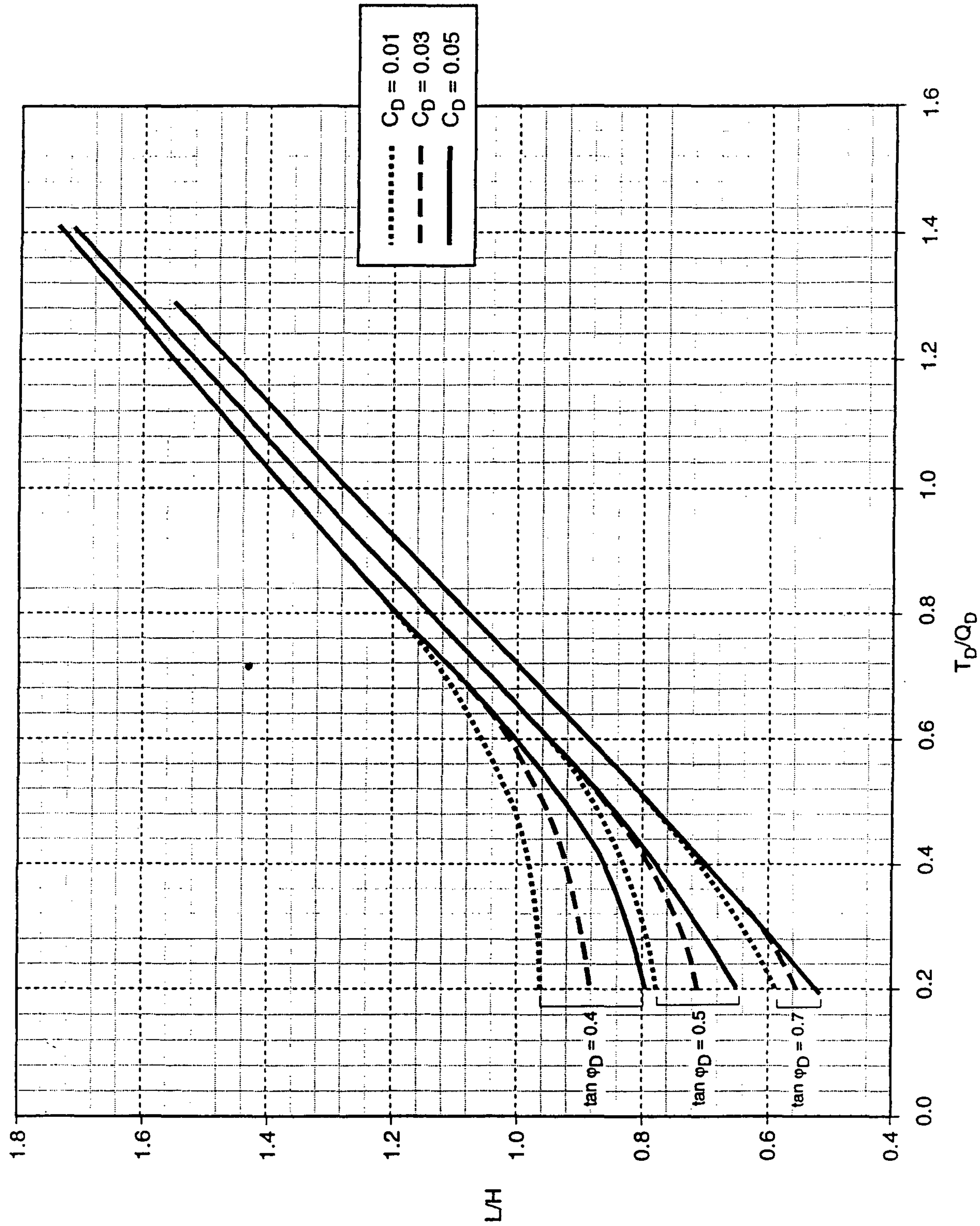

Figure 15. Design Chart Set 2: Backslope = 10°, Face Batter = 10° (Chart C).

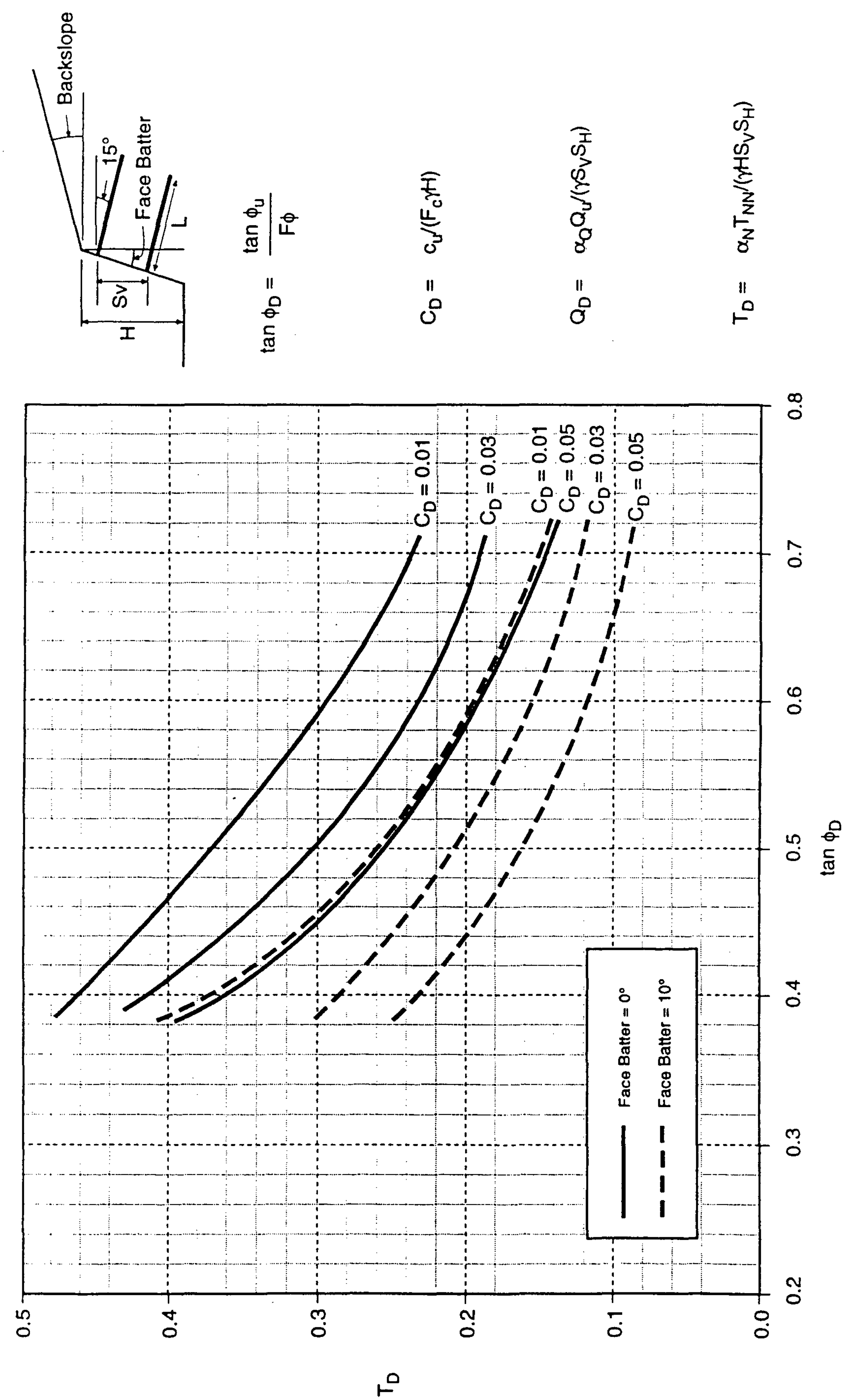

Figure 16. Design Chart Set 3: Backslope 20° (Chart A)

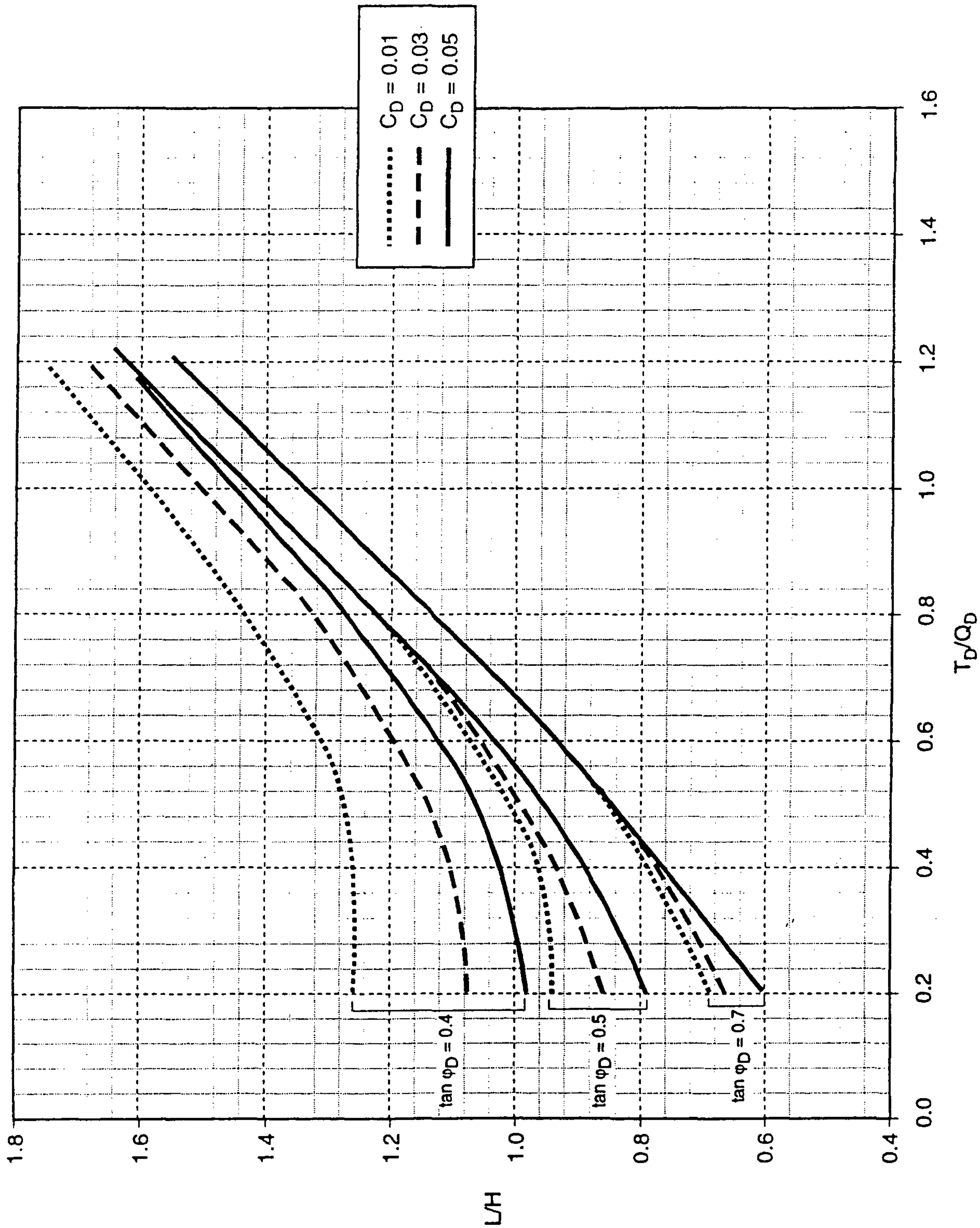

Figure 17. Design Chart 3: Backslope = 20°, Face Batter = 0° (Chart B)

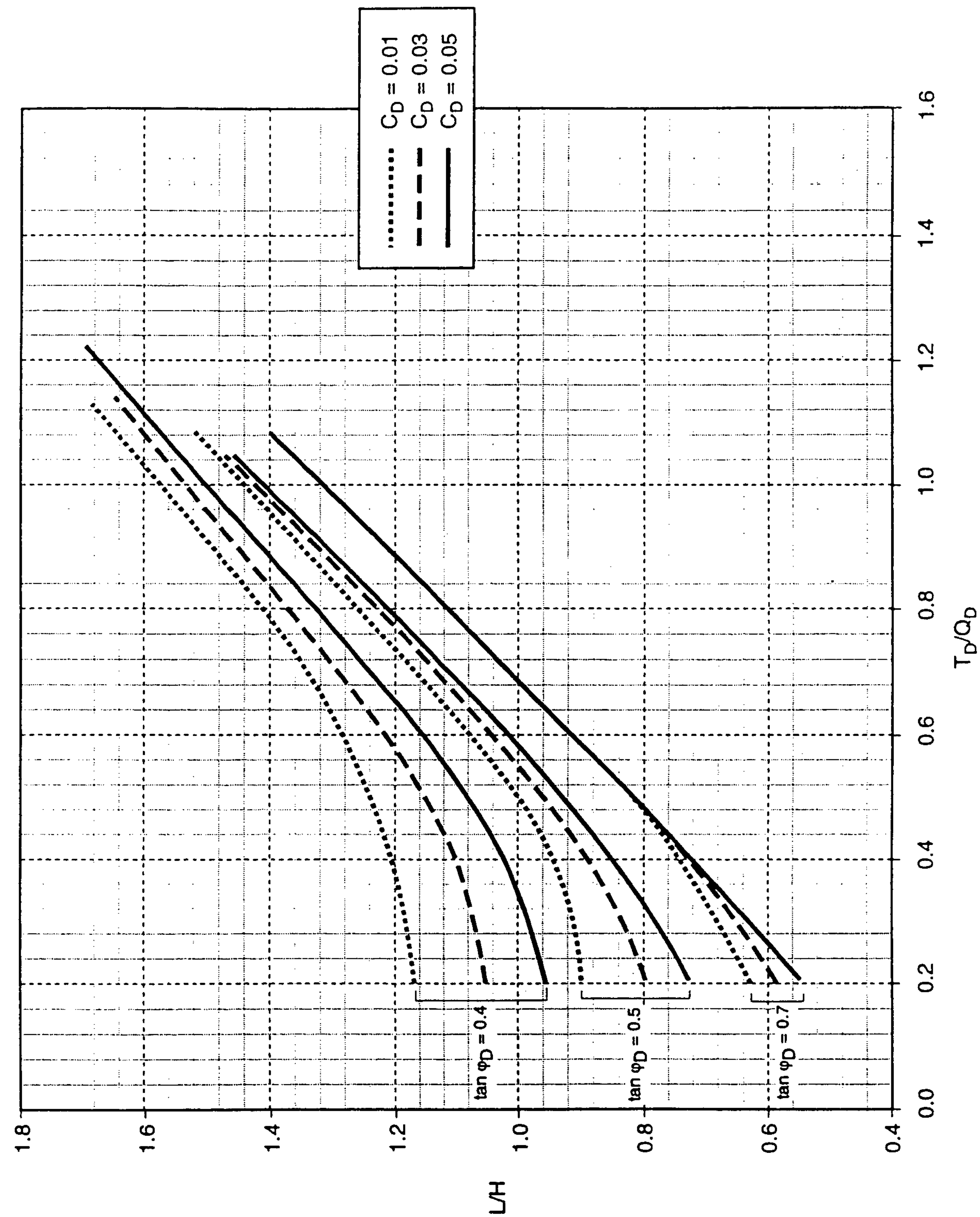

Figure 18. Design Chart 3: Backslope = 20°, Face Batter = 10° (Chart C)

Step 4

Compute the dimensionless nail pullout resistance. Divide the calculated dimensionless nail tensile capacity by the computed dimensionless nail pullout resistance, and determine the required nail length from the appropriate Chart B.

$Q_D = \alpha_Q Q_U/(\gamma S_V S_H) = (0.50)(60.0 \text{ kN/m})/[(18 \text{ kN/m}^3)(1.50 \text{ m})(1.50 \text{ m})]$

$Q_D = 0.74$

$T_D/Q_D = 0.23/0.74 = 0.31$

From Chart B (Figure 18), $L/H = 0.87$

$L = 0.87(9.50 \text{ m}) = 8.3 \text{ m}$

In summary, the design charts indicate a required bar yield strength of about 161 kN (use #25, Grade 420 bars) and a nail length of about 8.3 meters.

d. *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 40 mm to be achieved throughout the grout zone for nails that are not fully encapsulated. Centralizers should be placed at distances of 2.5 m center to center, with the lowest centralizer a maximum of 0.3 m from the bottom of the grouted drill hole.

2. In non-aggressive ground, the nail section could be resin-bonded epoxied using a 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 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 tiebacks, 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 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 40 mm will provide adequate protection. Centralizers must be provided. In aggressive ground full encapsulation should be considered.

e. Facing Design

To date, facings have been designed by either purely empirical methods based on experience, or by modeling the facing as a continuous two way slab/raft on an elastic foundation supported by the nails.

The nail forces have been computed by either considering the maximum tensile force (T_{max}) that can be carried by each nail or developed at working stress or by empirical relationships. Field data has documented a reduction of the maximum nail tensile load (T_{max}) at the face (T_o), as a function of nail spacing. The nail tensile force at the face has been approximated empirically from French research as:

$T_o/T_{max} = 0.5 + (S-0.5)/5$ for $1\ m \le S \le 3\ m$

$T_o/T_{max} = 0.6$ for $S \le 1\ m$

$T_o/T_{max} = 1.0$ for $S \ge 3\ m$

where S = the maximum horizontal or vertical spacing of the nails.

It has been recognized that methods used to date are quite conservative, and the newer method developed under FHWA-SA-96-069 allows for greater economy in design. For a detailed design, refer to FHWA-SA-96-069.

For permanent walls, the rough initial shotcrete face may be unsatisfactory for aesthetic reasons and 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 150 and 180 mm.

- **Separate Fascia Wall (CIP or Precast Panels).** As an alternative to the exposed shotcrete finish, the shotcrete 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 thick for constructability and shear stud connectors are welded to the nail cover plates to transfer load. A typical face design is shown on figure 19.

  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.

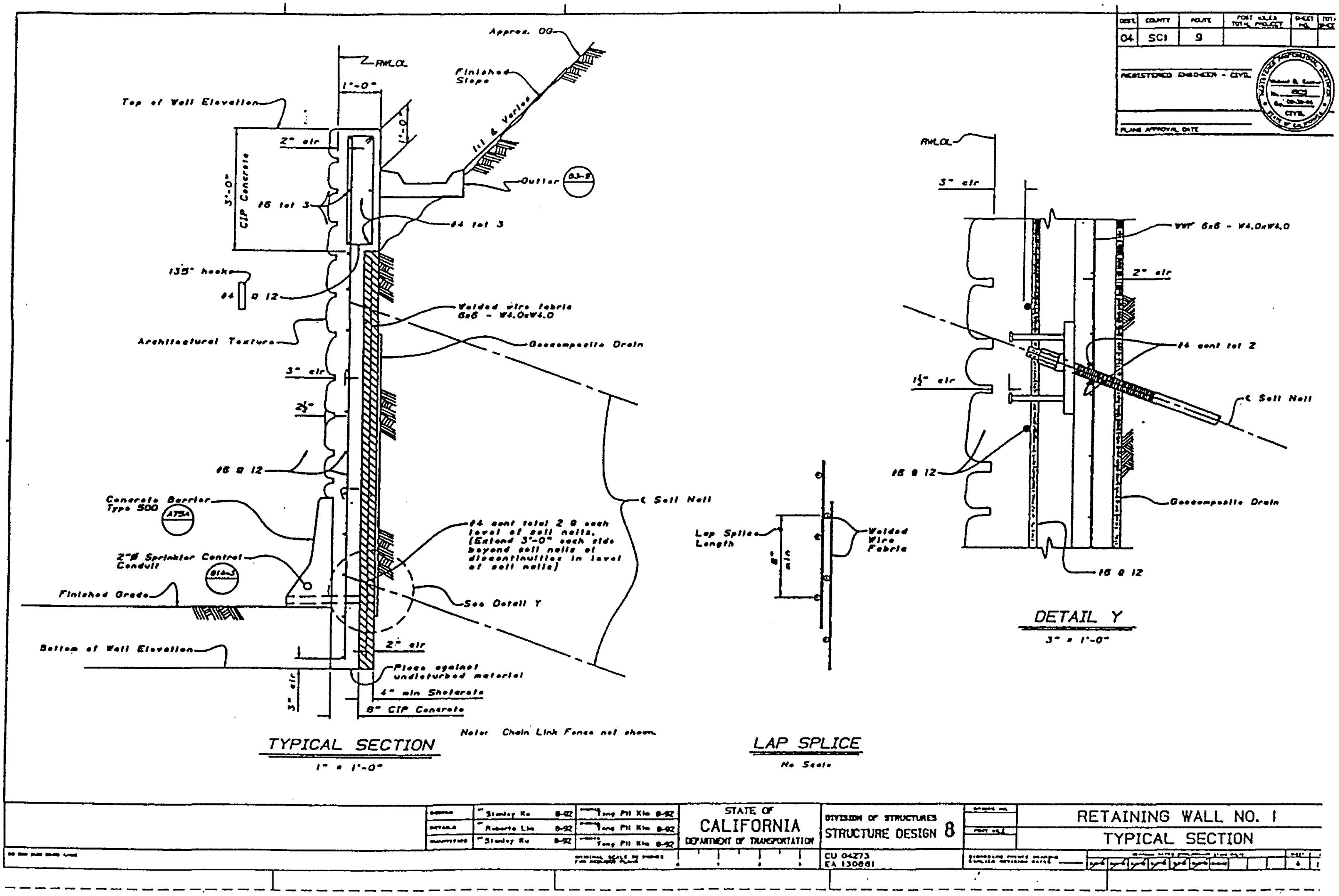

Figure 19. CALTRANS Details.

f. *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 geotextile drain mats can be placed vertically prior to applying shotcrete. Typical c-c spacings are 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:** By a shallow ditch behind the top of wall to lead away surface water. Ditch lining can be shotcrete or cast-in-place concrete.

- **Weep Holes and Horizontal Drains:** Fifty-mm diameter PVC pipe weep holes placed on approximate 3 m centers can be installed through the shotcrete face. These are plugged temporarily when shotcrete is applied.

  Longer 50-mm PVC horizontal drain pipes can also be installed in heavy seepage areas.

g. *Frost Protection*

The formation of ice lenses in the vicinity of the soil nail wall facing in frost-susceptible soils has been reported in a few cases. 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, and the availability of water. Increases in nail and facing loads should be anticipated in areas where frost durations are generally greater than one week and where there are frost susceptible soils near the face and in close proximity to a source of water. Frost loading effects may be eliminated or mitigated by the use of porous backfill (used on a few projects to date) or insulating material placed either between the shotcrete construction facing and the CIP or precast panel final facing, or outside the permanent concrete facing.

CHAPTER 5

BIDDING METHODS, CONSTRUCTION CONTROL AND CONSTRUCTION SPECIFICATIONS

5.1 INTRODUCTION

Two general types of contracting methods for soil nailed walls have been used:

- Method or Procedural Specifications with all details of design, construction materials and methods specified in the contracting documents.

  A variant to this method as often used in tieback 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.

- Performance or end result approach 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 over the other or a mixed use of approaches developed 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 it imposes on the owner, the construction control requirements, and 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.
- Submittals required by contractor, out lining drilling and grouting methods.
- Minimum drill hole diameters and length.
- Required soil nail design loads 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 strict 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 prequalification of contractors offers several benefits. Design of the structure is performed by trained and experienced staff and can utilize the contractor is 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 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 at first understand the technology and therefore not be fully qualified to review and approve construction modifications.

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 is contained as complete special provisions, as would construction control and monitoring requirements.

These plans furnished as part of the contract documents 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 qualify methods that the contractor uses as part of its prequalification process.

- Provide or make available the results of the geotechnical investigation.

- Specify submittals which the contractor must provide, including calculations, drawings and construction methods.

- Specify 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 little or no experience with this technology or for complicated projects where a specialty contractor specific or local knowledge can be used to the best advantage.

5.4 CONSTRUCTION CONTROL

Prior to construction of the soil nailed 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.

- Prequalification 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 redrilling. 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.

- Assurance of adequate shotcrete strength, thickness, and proper placement of reinforcement in the shotcrete facing.

- 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.

- Proper installation of deep drains, weepholes, and prefabricated vertical drains. It is essential that hydraulic continuity of the vertical drains is assured if installed incrementally.

Nail Testing

Nail testing in 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 and a non-grouted zone of approximately 1 m 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.

More detailed requirements for inspection and QA/QC testing are fully outlined in FHWA SA-93-068 "Soil Nailing; Field Inspectors Manual."

5.5 CONSTRUCTION SPECIFICATIONS FOR PERMANENT SOIL NAILED WALLS

The recommended construction specifications with commentary are fully developed in FHWA-SA-96-069.

CHAPTER 6

CASE HISTORIES

6.1 SOIL NAILED ABUTMENT I-495, VIRGINIA[5]

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 20 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.

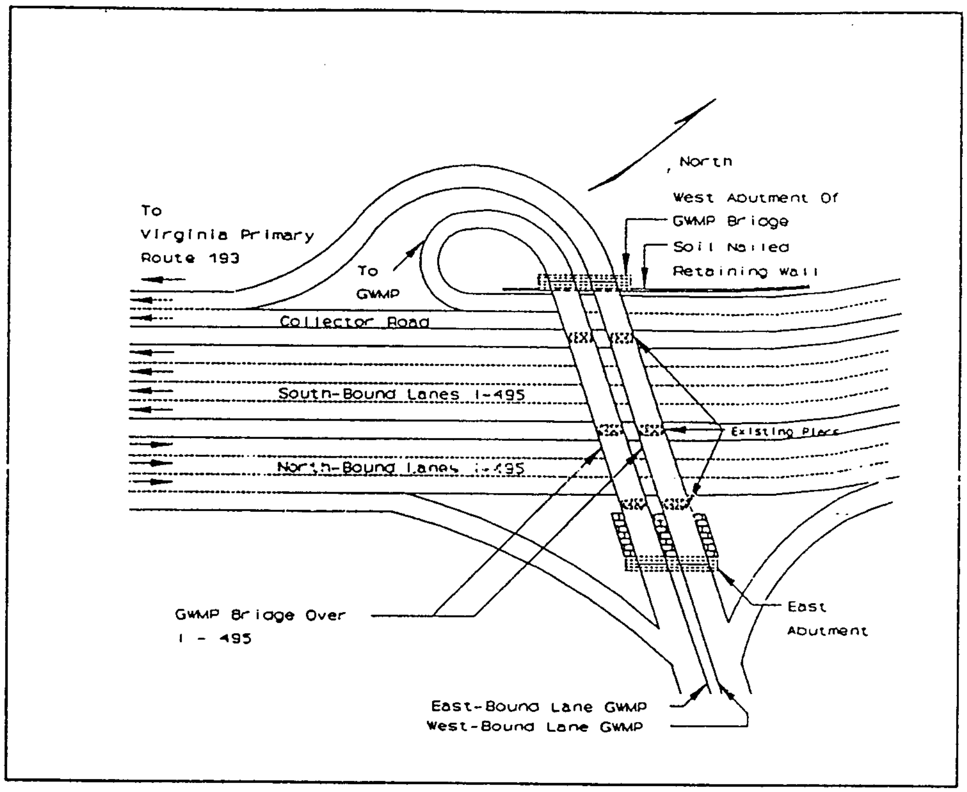

Figure 20. Plan view of project location after construction of Soil Nailed Retaining Wall.

The contract plans required the construction of a steel H-pile and timber lagging tieback 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 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 $\gamma = 18.8$ KN/m^3 was used throughout this design. A design value of $\phi = 28°$ was used for the angle of internal friction and $c = 0$ for the cohesion of the soil. In general, qualitative terms, the soil is 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 to 50. The assumed strength values are extremely conservative as reported extensive testing in Piedmont residual soils usually yields ϕ values of 30 to 32 degrees. Consistent with the above, an adhesion of 88 KPa 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 in length. Of this, approximately 33 m 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. 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. Global stability using a "Modified Davis" Method as outlined in FHWA RD89-198, determined minimum nail lengths and spacing for the portion of wall not subject to abutment loading and a starting point for nail lengths and spacing beneath the abutment.

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 horizontal and 1.22 m vertical and a 200 mm diameter grouted hole and is shown on figure 21. Note that the top 3 rows of nails are longer, with an L/H ratio of 1.0 than the bottom 2 rows with an L/H ratio of 0.8.

For the balance of the wall, a spacing of 1.5 m horizontal and vertical was used with a smaller 130 mm grouted hole. The resulting uniform lengths were consistent with a 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, which was to be placed in two layers, was designed for this structure. Reinforcement at each soil nail was provided by a 600 mm square grid of #4 reinforcement steel secured on 100-mm centers with 50 mm of cover provided between the excavated soil face and the reinforcing grid. After the first layer of shotcrete was placed, a 200 mm x 200 mm x 10 mm 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 centers. Again, a minimum of 50 mm of cover was specified from the outside of the facing to the reinforcing mesh, and this was provided by the second layer of shotcrete. 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 with #4 reinforcement steel placed on 300-mm centers in both the horizontal and vertical directions. A minimum cover of 50 mm was specified for this reinforcement steel. As this CIP fascia is merely an architectural veneer and is nonstructural, the reinforcement is 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.

Details are shown in figure 21 and 22.

Monitoring

Because of the walls experimental designation, a monitoring program to verify key design assumptions and monitor horizontal movements was implemented.

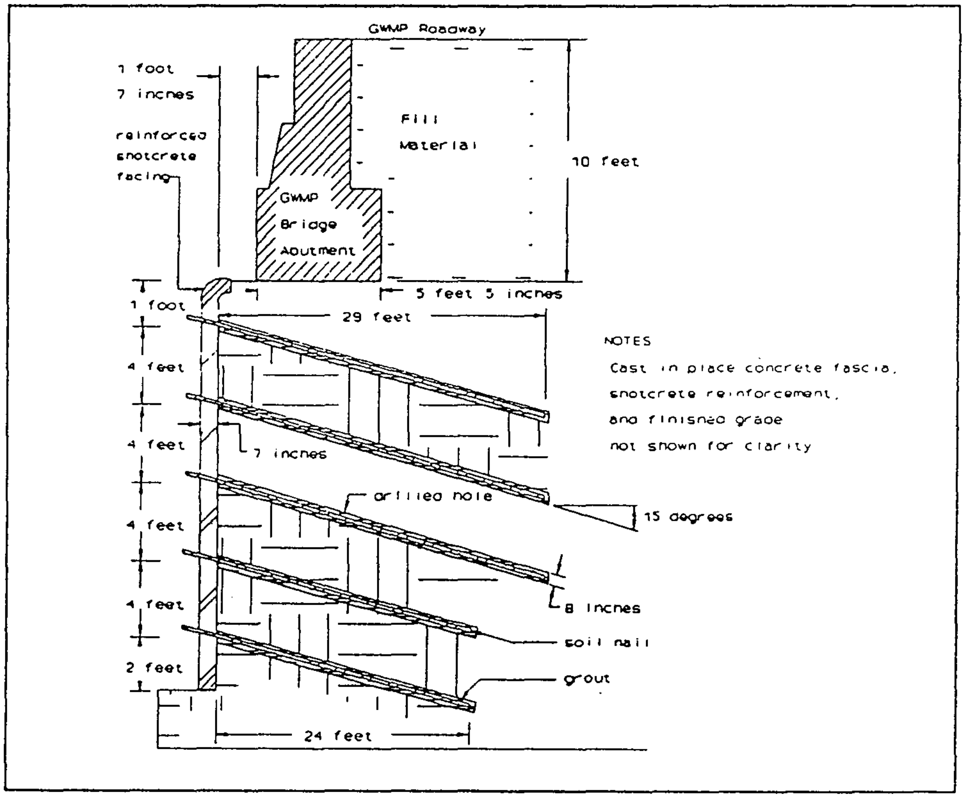

Figure 21. Typical section of Soil Nailed Retaining Wall under GWMP Bridge Abutment (Not to scale).

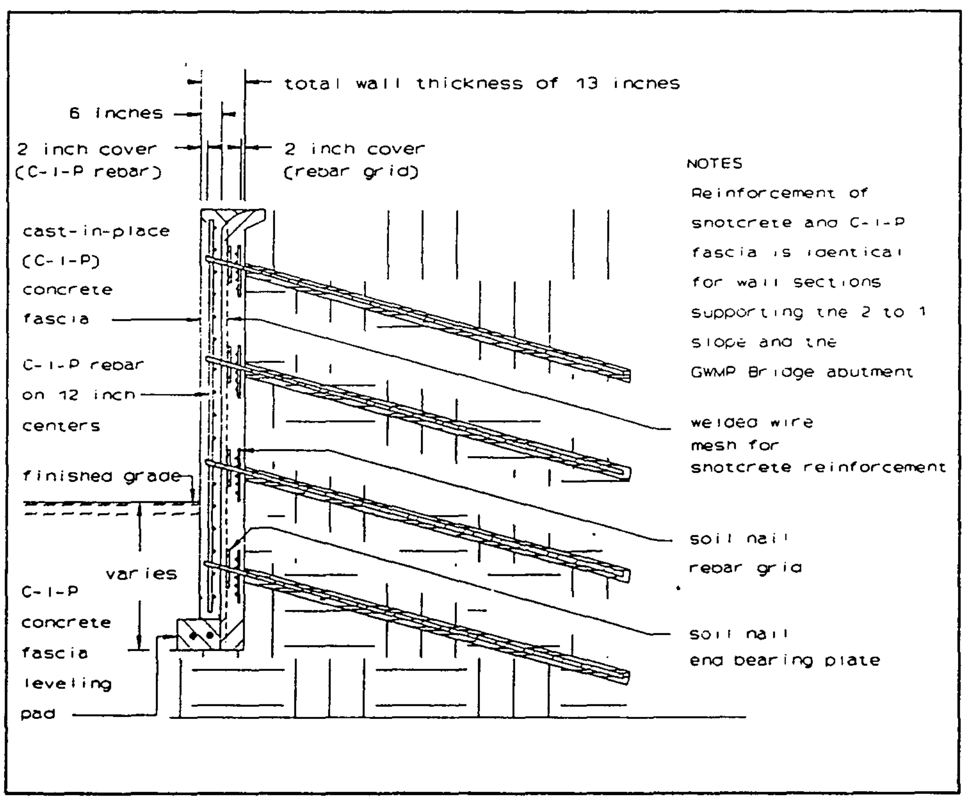

Figure 22. Typical Section showing reinforcement for CIP fascia and shotcrete (Not to scale).

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. All test nails achieved the required capacity and typically developed a load/deflection similar to the one shown on figure 23. 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 to 0.16 percent 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.

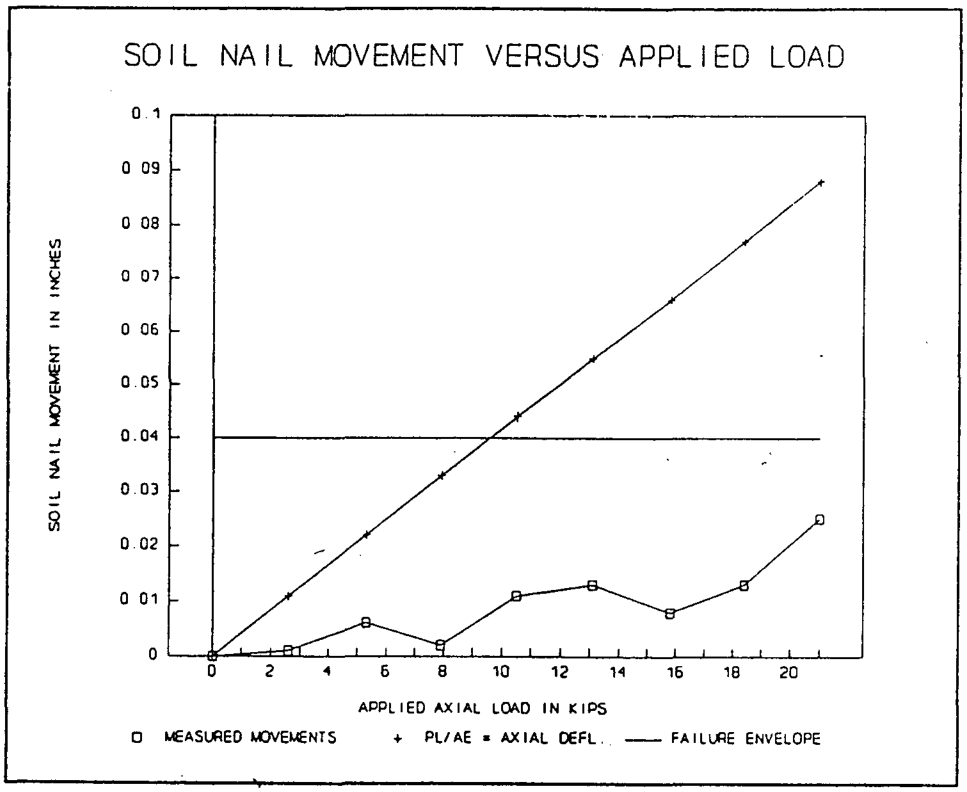

Figure 23. Pullout test results.

6.2 SOIL NAILED WALL, TONAWANDA DRIVE, SAN DIEGO, CA[6]

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 to 8 m supporting a broken back soil surcharge

(2:1). The design required the installation of one to two rows of tiebacks at 2.44 m centers, between two W8 x 48 soldier piles encased in a 760 mm diameter concrete shaft, extending 5.5 to 11 m 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 indicate 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 $\gamma = 18$ kN/m^3, a ϕ of 33°, a cohesion of 24 kPa, and an soil/grout allowable adhesion of 104 kPa consistent with a 150 mm drilled nail hole diameter.

Design Development

The soil nail plans for the 153 m long wall were developed based on a horizontal nail spacing of 1.6 m. The resulting nail lengths varied from 5.5 to 6.1 m or having a L/H ratio of 0.76, with a structural nail crossectional 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 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 and provided for a savings of \$268/m^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 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, which increased the capacity sufficiently to meet the requirements of the design proposal.

The completed construction is shown in figure 24.

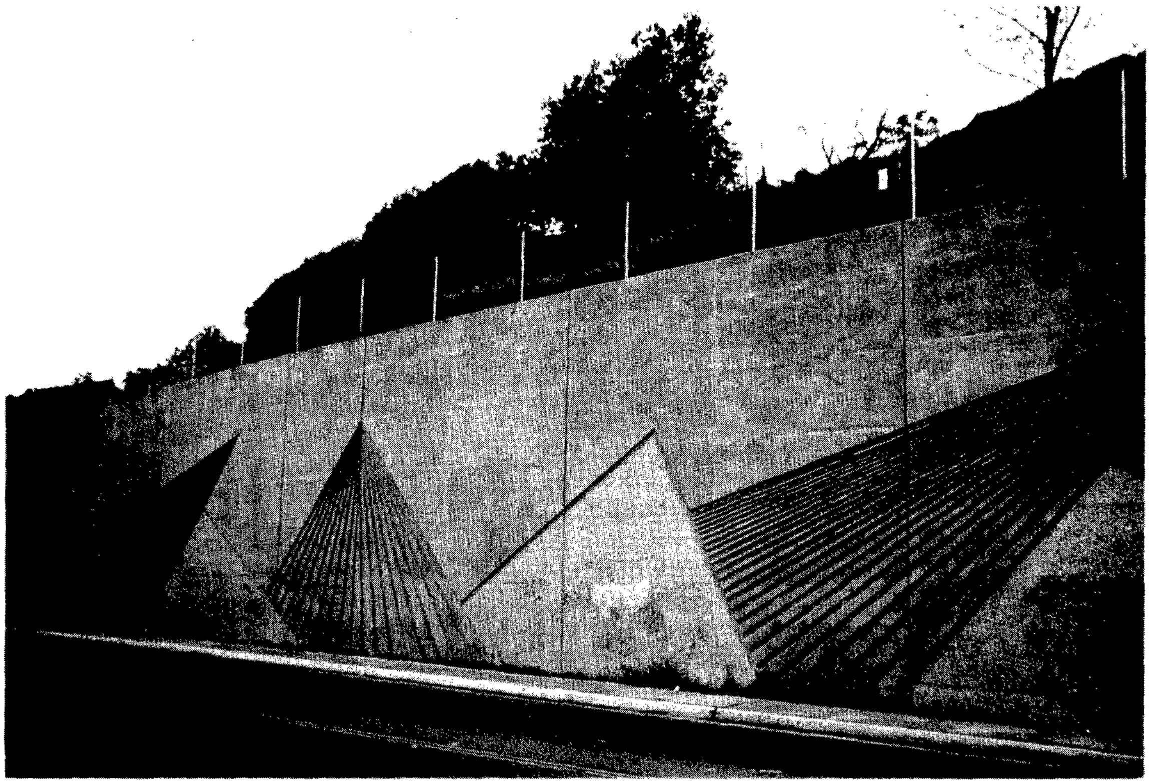

Figure 24. Completed Tonawanda Wall.

CHAPTER 7

COST DATA

7.1 PERMANENT STRUCTURES

Table 7 summarizes historical bid data compiled under FHWA-SA-96-069.

Table 7. US 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
RT 85 Saratoga, CA	1992	1,732	CIP	$380

Facing Type Key - CIP - cast-in-place; S - shotcrete; SP - shotcrete, precast panels

Project	Year	Wall M^2	Facing Type	Bid/M^2
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

Facing Type Key - CIP - cast-in-place; S - shotcrete; SP - shotcrete, precast panels

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. Architectural finishes usually add $30/m^2. Precast panel or timber faced walls averaged $600/m^2.

7.2 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 to $400/m^2.

The difference in cost is based on the cost of the permanent facing (200+ mm of cast-in-place concrete or precast panels) and corrosion protection for nails.

REFERENCES

1. Weatherby, D.E. (1982), "Tiebacks" FHWA RD 82-047
   Federal Highway Administration, Washington D.C.

2. Louis, C., (1986) "Theory and Practice in Soil Nailing Temporary or Permanent Works"
   ASCE Geotech Conference, Boston

3. EM 1110-2-2005 (1982) - Standard Practice for Shotcrete.
   Department of the Army, Corps of Engineers Office of the Chief of Engineers.

4. ACI - 506.66 (Revised 1983) *American Concrete Institute,* Detroit, Michigan.

5. Graham, D., J. (1991) "Construction Report for a Permanent Soil Nailed Retaining Wall to Support an Existing Bridge Abutment's Spread Footing Foundations"
   FHWA, Region 3, Office of Structures.

6. Private Communication. R. Chassie and D. Weatherby. (1995)

7. R. Chassie, J. Keeley (1993). "Soil Nailing Overview", *Tenth Annual International Bridge Conference*, Pittsburgh, PA.