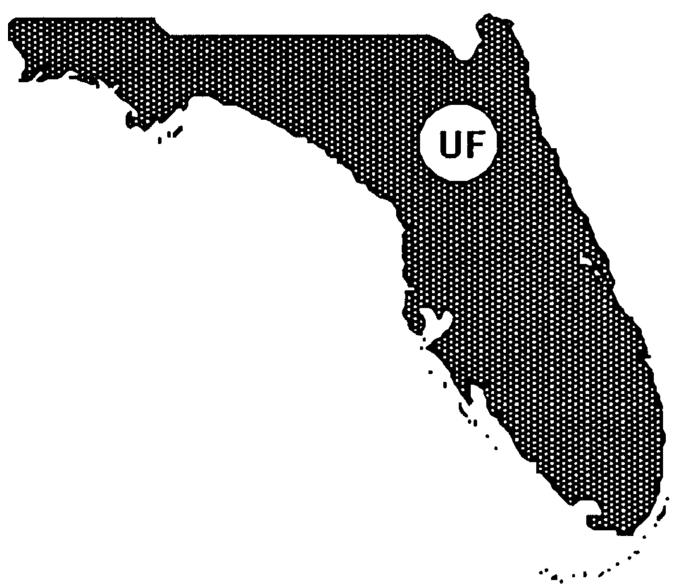
FINAL REPORT

A Compendium of Ground Modification Techniques

Submitted by
F.C. Townsend
J. Brian Anderson



Submitted to: **Florida Department of Transportation**

Agency ID Nos. BC 354, RPWO# 64

UF Contract No. 4910-4504-887

April 2004

Technical Report Documentation Page

1. Report No.	2. Government Accession No.	3. Recipient's Catalog No.
Final Report		

4. Title and Subtitle	5. Report Date
A Compendium of Ground Modification Techniques	April 2004
	6. Performing Organization Code
7. Author(s) F.C. Townsend and J. Brian Anderson	8. Performing Organization Report No. 4910-4504-887
9. Performing Organization Name and Address University of Florida Department of Civil & Coastal Engineering 124 Yon Hall / P.O. Box 116580 Gainesville, FL 32611-6580	10. Work Unit No. (TRAIS) 11. Contract or Grant No. BC 354, RPWO# 64
12. Sponsoring Agency Name and Address Florida Department of Transportation Research Management Center 605 Suwannee Street, MS 30 Tallahassee, FL 32301-8064	13. Type of Report and Period Covered Final Report Nov. 16, 2001 – April 30,2004 14. Sponsoring Agency Code

15. Supplementary Notes

Prepared in cooperation with the Federal Highway Administration

16. Abstract

Increased growth has lead to the need to use marginal sites, and because many soils can be made into useful construction material when properly modified, ground modification has become a viable consideration for many infrastructure projects. This report summarizes the advantages / limitations, appropriate selection of ground modification technique, and design considerations for the following ground modification techniques:

1. Compaction
2. Deep Dynamic Compaction
3. Surcharging (preloading)
4. Stone Columns
5. Soil Stabilization
6. Grouting
7. Jet Grouting
8. Deep Soil Mixing
9. MSE Walls
10. Reinforced Slopes
11. Soil Nailing
12. Geogrids in Highways
13. Lightweight Fills

17. Key Words	18. Distribution Statement
Ground Modification, Soil Improvement, Deep Dynamic Compaction, Prefabricated Vertical Drains, Lightweight Fills, Stone Columns, Grouting, Vibro-Floatation	No restrictions. This document is available to the public through the National Technical Information Service, Springfield, VA, 22161

19. Security Classif. (of this report)	20. Security Classif. (of this page)	21. No. of	22. Price
Unclassified	Unclassified	350	

Form DOT F 1700.7 (8-72)
Reproduction of completed page authorized

TABLE OF CONTENTS

Page

CHAPTER

LIST OF TABLES

Page

TABLE

LIST OF FIGURES

Chapter 1

INTRODUCTION TO GROUND MODIFICATION METHODS

Because increased growth has lead to the need to use marginal sites, and because many soils can be made into useful construction material when properly modified, ground modification has become a viable consideration for many infrastructure projects. (Mitchell 1981). When difficult conditions are encountered, possible solutions are: (Hausmann 1990, Mitchell 1981)

1. Avoid the site by relocation or abandonment.
2. Design the planned structure to resist the unfavorable conditions. Some examples are; to use a deep foundations system to transfer surface loadings to deeper more competent strata, design a very stiff structure that will tolerate settlements, or use very flat slopes.
3. Remove and replace unsuitable soils. In these cases, unfavorable organics, soft clays, swelling soils that are compressible or expansive are removed and replaced by more suitable soils. This technique is common practice in highway construction.
4. Use ground modification techniques to improve the on-site soils and conditions.

GROUND MODIFICATION METHODS

Currently, viable ground modification methods can be categorized as:

1. Compaction. The soil unit weight is increased by short-term applications of mechanical energy. Compaction of surface layers via sheepsfoot, vibratory, static, and pneumatic tired rollers is the most widely used ground modification method. Compaction of cohesionless soils by deep dynamic compaction using heavy tampers, or vibrating probes is also a commonly used method.
2. Consolidation by use of preloading and/or vertical drains.
3. Grouting involving the pressure injection of cementing or waterproofing agents.
4. Soil stabilization by using cementatious admixtures mixed with near surface soils and subsequently compacted.
5. Soil reinforcement by use of strengthening inclusions. The reinforcement by inclusion of metal or plastic strips, bars, geotextiles, add tensile and compressive reinforcement to soil masses. Slope stability improvements via reinforced slopes, and soil nailing are potential methods. Stone columns also provide increased compressive and shear resistance.
6. Load reduction via use of lightweight fills (Styrofoam, shredded tires) can minimize potential settlement problems of embankments on soft foundations.

Selection of Ground Modification Method (Mitchell 1981, Hausmann 1990 ,and US Army Corps of Engineers 1999).

The selection of the most suitable ground modification method for a given project can only be made after evaluation of several factors pertinent to the geotechnical problem. Among these are:

1. The nature of the problem; i.e., settlement, slope improvement, bearing capacity, etc. Type and degree of modification required.
2. The area, depth, and total volume of the soil to be treated.
3. Soil type and its initial properties.
4. Availability of equipment and materials.
5. Time available and cost.
6. Environmental constraints including effects of adjacent structures and property.

Table 1.1 (US Army Corps of Engineers 1999) summarizes potentially applicable ground modification methods, while Figure 1.1 (Mitchell 1981) relates various methods to the range of grain-sizes for which it is most applicable.

An important factor in selection of a suitable ground improvement method is the accessibility of the site, particularly if the site is already developed. When ground improvement is needed on large, open and undeveloped sites, there are typically more and less expensive options available than at sites that are small or have constraints such as existing structures or facilities.

A brief description of each of the methods is given below. More detailed discussions may be found in Mitchell (1981), FHWA (1983, 1986, 1986c, 1996a 1996b, 1998), Hausmann (1990), Mitchell and Christopher (1990), Hayward Baker (1996), and ASCE (1997).

COMPACTION

Deep Dynamic Compaction (US Army Corps of Engineers 1999)

Deep dynamic compaction (DDC), also called heavy tamping, consists of repeated dropping of heavy weights onto the ground surface to densify the soil at depth, as shown in Figure 1.2 . For unsaturated soil, the process of DDC is similar to a large-scale Proctor compaction test. For loose, fully saturated, cohesionless soils, the impact from the weight liquefies the soil and the particles are rearranged in a denser, more stable configuration. At developed sites, a buffer zone around structures of about 30 to 40 meters is required. A typical DDC program involves weights of 10 to 30 tons dropped from heights of 15 to 30 meters at grid spacings of 2 to 6 meters. DDC works best on sands and silty sands, with a maximum effective densification depth of about 10 meters. The maximum improvement occurs in the upper two-thirds of the effective depth. The relationship between the effective depth, the weight and the height of the drop can be expressed as:

$$D = (0.3 \text{ to } 0.7)*(WH)^{1/2}$$

where D = maximum depth of improvement, m

W = falling weight, metric tons

H = height of drop, m.

Table 1.1 Summary of Ground Modification Methods (from US Army Corps of Engineers 1999)

Method	Soil Type	Typical Spacing	Attainable Improvement	Advantages	Limitations	Costs
1) Compaction	All soils	—	Std. density 95+%	Low cost, simple	Equipment – soil compatability, field control	Low
2) Deep Dynamic Compaction (DDC)	Saturated sands and silty sands; partly saturated sands	Square pattern, 2 to 6 m spacing	D_r = 80% $(N_1)_{60}$ = 25 q_{c1} = 10-15 MPa	Low cost, simple	Limited effective depth (10 m), clearance required, vibrations	Low ≈ $5/m³
3) Vibrocompaction	Sands, silty sands, gravelly sands < 20% fines	Square or triangular pattern, 1.5 to 3 m spacing	D_r = 80+% $(N_1)_{60}$ = 25 q_{c1} = 10-15 MPa	Proven effectiveness, uniformity with depth Depths ≈ 20 m	Special equipment, unsuitable in cobbles and boulders, ineffective above 3 m	Low to moderate $1-$4 / m³
4) Surcharging and use of prefabricated vertical (PV) Drains (Wick Drains)	Moderately to highly compressible soils; clayey sands, silts, clays and their mixtures	Square or triangular pattern, spacing 1.5 to 6 m	Depends on final consolidation pressure	Proven effectiveness, low cost, simple, computer software	Unsuitable if obstructions exist above compressible layer, time	Drains only $2-$4 / lin m.
5) Penetration Grouting	Sands and coarser materials	Triangular pattern, 1 to 2.5 m spacing	Void filling and solidification	No excess pore pressure or liquefaction, can localize treatment area, unlimited depths	High cost, fines prevent use in many soils	Moderate – High $3-$30 / m³
6) Compaction grouting	Any rapidly consolidating, compressible soil including loose sands	Square or triangular pattern, 1 to 4.5 m spacing, with 1.5 to 2 m typical	Up to D_r = 80+% $(N_1)_{60}$ = 25 q_{c1} = 10-15 MPa (Soil type dependent)	Controllable treatment zone, useful in soils with fines	High cost, post-treatment loss of prestress	Moderate – High $5-$50/m³
7) Jet grouting	Any soil; more difficult in highly plastic clays	Depends on application	Solidification of the ground – depends on size, strength and configuration of jetted elements	Controllable treatment zone, useful in soils with fines, slant drilling beneath structures	High cost	High $200 - $650 /m

Table 1.1 Continued

Method	Soil Type	Typical Spacing	Attainable Improvement	Advantages	Limitations	Costs
8) Replacement	All soils	N/A	High density fills to cemented materials	Can design to desired improvement level	Expensive, might require temporary support of existing structures, Depths ≈ 2-3 m	----
9) Admixture Stabilization	Cement – sands and silty sands Lime – clays and clayey sands	N/A	High density fills to cemented materials	Can design to desired improvement level	Results depend on degree of mixing & compaction achieved in field, Depth ≈ 2 m	Low – Moderate ≈\$2-\$4 / m^3
10) MSE Walls	Clean cohesionless backfill	Continuous horizontal geogrids, Metal strips 2-3 m spacings	N/A	Excellent walls, well known computer software	Wall heights 3-15 m, ROW behind wall	\$225 / m^2 wall face
11) Reinforced Slopes	Clean cohesionless backfill	Continuous horizontal geogrids	Slopes up to 2V:1H	Well known computer software	ROW constructing slope erosion protection of face	Moderate – High ≈\$45 / m^3
12) Soil Nailing	Any drillable soil, except very soft clays	1 grouted nail per 1 to 5 m^2, 1 driven nail per 0.25 m^2	Stabilize cut slopes and excavations	Flexible system, can tolerate large movements, highly resistant to dynamic loading, can install with small, mobile equipment, reinforcement is redundant, so weak nail will not cause catastrophic failure, computer software	Excavation or cut slope must remain stable until nails are installed, difficult to construct reliable drainage systems, may require underground easement on adjacent property	Moderate – High \$165-\$775/m^2 face

Table 1.1 Continued

Method	Soil Type	Typical Spacing	Attainable Improvement	Advantages	Limitations	Costs
13) Stone columns (Vibro-replacement)	Soft, silty or clayey sands, silts, clayey silts	Square or triangular pattern, 1.5 to 3 m center to center column spacing	$(N_1)_{60}$ = 20 q_{c1} = 10-12 MPa	Proven effectiveness, drainage, reinforcement, uniformity with depth, bottom feed dry process puts fill where needed	Special equipment, can not use in soil with cobbles and boulders, Depth ≈ 20 m	Moderate $45-$60/m
14) Geogrids	Base course	1 to 2 grids	Reduction of base thickness by ½	Simple, cost effective	Granular base course	Moderate $20 /$m^2$
15) Light-weight fills	Soft compressible soils	N/A	Elimination of undesirable settlements	Simple	Tire shreds compressible special construction	Moderate - High
16) Deep soil mixing	All soils	Select treatment pattern depending upon application	Depends upon size, strength and configuration of DSM elements	Positive ground reinforcement, high strength	Requires special equipment, Depth ≈ 20 m	High – V. High

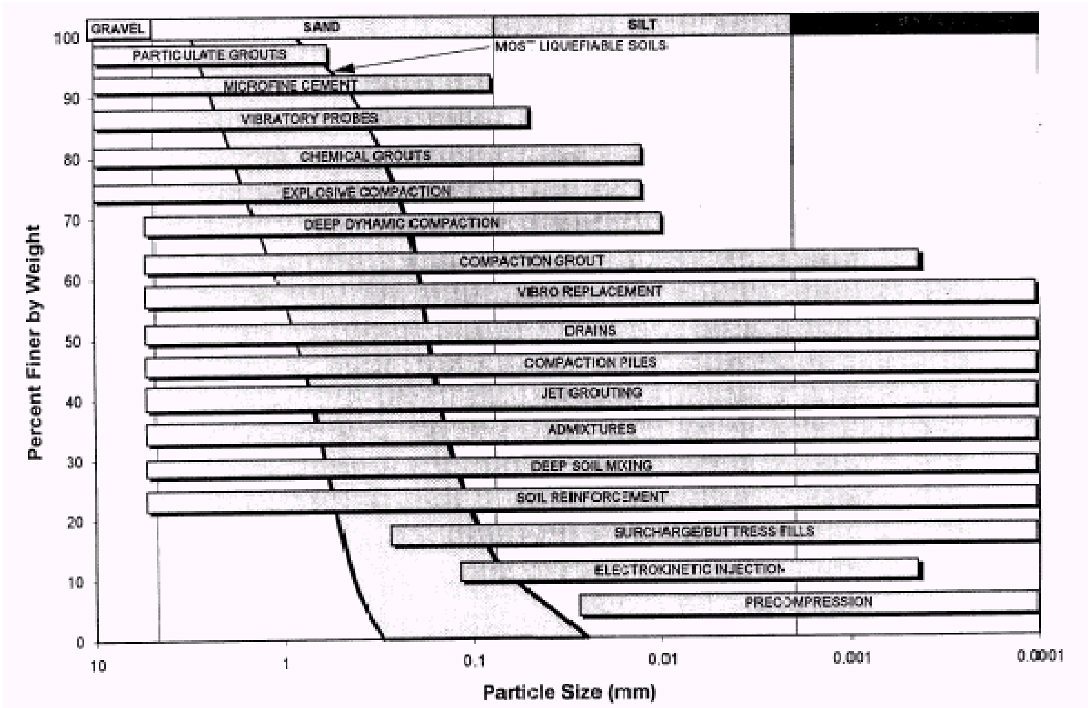


Figure 1.1 Applicable Grain Size Ranges for Soil Improvement Methods

(from Mitchell, 1981)

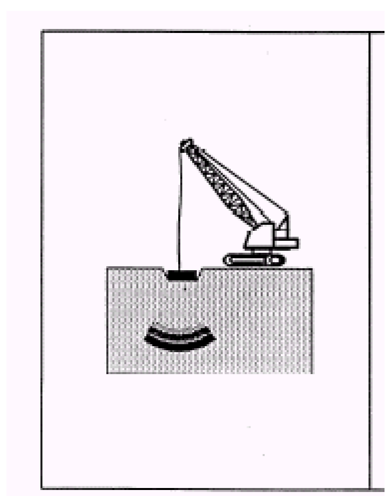


Figure 1.2 DDC Process

The lower values for the coefficient generally apply to silty sands, whereas, clean coarse, cohesionless soils are densified to a greater effective depth for a given value of W*H. DDC is discussed in greater detail in Mitchell (1981), FHWA (1986a), and Hayward Baker (1996).

Vibrocompaction (US Army Corps of Engineers 1999)

Vibrocompaction methods use vibrating probes (typically having a diameter of about 0.4 m) to density the soil. A sketch showing the vibrocompaction process in shown in Figure 1.3. The probe is usually jetted into the ground to the desired depth of improvement and vibrated during withdrawal, causing densification. The soil densifies as the probe is repeatedly inserted and withdrawn in about 1 m increments. The cavity that forms at the surface is backfilled with sand or gravel to form a column of densified soil. Vibrocompaction methods are most effective for sands and gravels with less than about 20 percent fines, as shown in Figure 1.4.

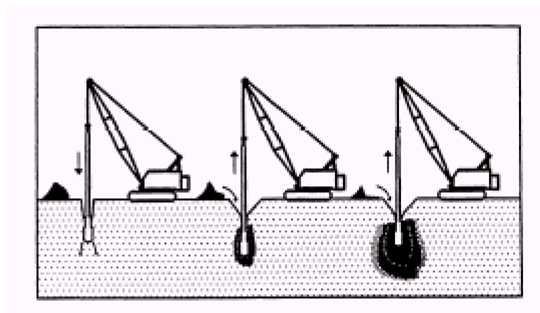


Figure 1.3 The Vibrocompaction Process (Hayward Baker, 1996)

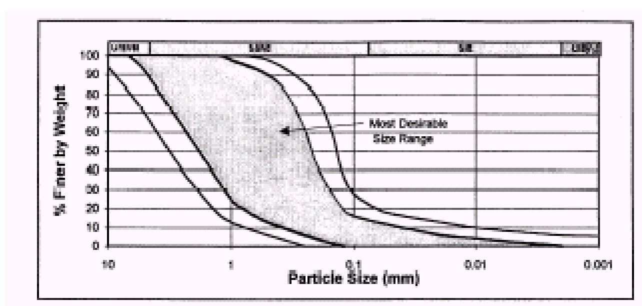


Figure 1.4 Range of Particle Size Distributions Suitable for Vibrocompaction

When vibrocompaction is used for large areas, it is typically performed using either a triangular or rectangular grid pattern with probe spacings in the range of 1.5 m to 3 m on centers. The spacing depends on several factors, including the soil type, backfill type, probe type and energy, and the level of improvement required. While field tests are usually done to finalize the design. Advantages of vibrocompaction are that the vibrations felt on or near the site are significantly less than caused by deep dynamic compaction or explosive compaction and more uniform densification is obtained. On the other hand, the cost is usually greater. Additional information is available in Mitchell (1981), Hausmann (1990), and Hayward Baker (1996).

Prefabricated Vertical (PV) Drains, With or Without Surcharge Fills (US Army Corps of Engineers 1999)

Prefabricated vertical (W) drains, also known as wick drains, are typically installed in soft, cohesive soil deposits to increase the rate of consolidation settlement and corresponding strength gain. The rate of consolidation settlement is proportional to the square of the length of the drainage path to the drain. Installing vertical drains shortens the drainage path, which causes an increase in the rate of settlement. Geocomposites are widely used as drains because they are relatively inexpensive, economical to install and have a high flow capacity. Geocomposite drains consist of a plastic waffle core, which conveys the water and a geotextile filter to protect the core from clogging. In selecting a drain it is important to choose one with enough capacity. Drains are typically spaced in a triangular or rectangular configuration. A sand blanket is usually placed on the surface of the consolidating layer to facilitate drainage. For additional information on engineering assessment and design of vertical drains, the 1986 FHWA publications titled ***Prefabricated Vertical Drains*** and ***Geocomposite Drains*** may be consulted. A discussion of the updates in PV drains in the past ten years can be found in ASCE (1997).

Surcharge preloading can be used in conjunction with vertical drains to increase the magnitude of settlement prior to construction, as shown in Figure 1.5. Surcharge preloading consists of placing a surcharge load over the footprint of the proposed facility prior to construction. The surcharge load causes consolidation settlement to occur.

Penetration Grouting (US Army Corps of Engineers 1999)

Penetration grouting is a process by which the pore spaces in soil or the joints in rock are filled with grout, as depicted in Figure 1.6. Injection pressures are usually limited to prevent fracture or volume change in the formation. One rule of thumb for maximum injection grouting

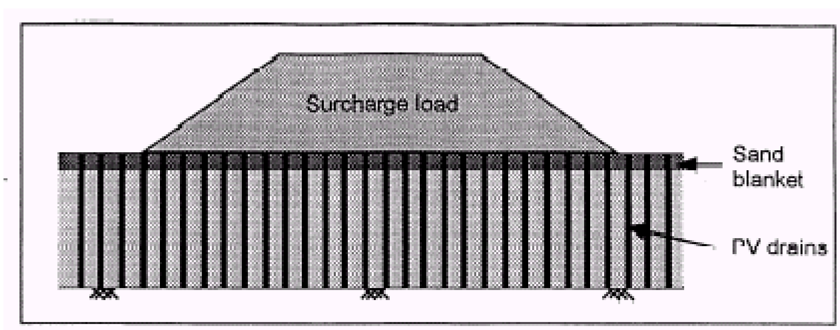


Figure 1.5 PV Drains With Surcharge Load

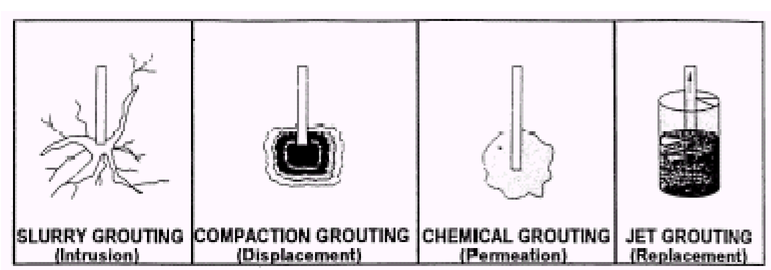


Figure 1.6 Types of Grouting (Hayward Baker, 1996)

pressures is 20 kPa per meter of depth (1 psi/ft). Either particulate or chemical grouts can be used. The process is limited to relatively coarse-grained soils, because the grout must be able to flow through the formation to replace the fluid in the void spaces or joints. Particulate grouts, such as cement or bentonite, are used for soils no finer than medium to coarse sands, since the particles in the grout must be able to penetrate the formation. Use of micro-fine cement enables penetration of somewhat finer-grained soil than can be treated using ordinary Portland cement. Chemical grouts, usually silicates, can be used in formations with smaller pore spaces, but are still limited to soils coarser than fine sands. The typical spacing for penetration grouting holes is between about 4 to 8 feet. For water cutoff applications, two or three rows of grout holes are usually required to form an effective seepage barrier. Penetration grouting can also be used for ground strengthening. Whereas seepage control requires essentially complete replacement of the pore water by grout, effective strengthening is possible with incomplete replacement. Additional references on permeation grouting include Karol (1990) and Xanthakos et al. (1994).

Compaction Grouting (US Army Corps of Engineers 1999)

Compaction grouting consists of injecting a very-low slump mortar into loose soils and cavities. The grout forms a bulb, which expands against the surrounding soil, causing densification and displacement to occur (Figure 1.6). Unlike penetration grouting, the grout does not penetrate the soil pores in compaction grouting. The grout acts as a radial hydraulic jack to compress the surrounding soil. The grout is usually a mix of sandy soil with enough fines to bind the mix together, cement, and water. A typical compaction grout mix consists of about 3 parts sand to 1 part cement, although cement is not always used. The grout forms a bulb up to about 1 m in diameter that is relatively strong and incompressible after it hardens. The process causes an overall decrease in the void ratio of the formation. Compaction grouting is most effective for loose granular soils, collapsible soils, and loose, grained soils. A typical compaction-grouting program consists of pipe spacings between 3 to 15 feet, with 5 to 7 feet spacing common. The pumping rate may vary from 0.5 to 10 cubic feet per minute, depending on the type of soil being treated. The replacement factor, which is the percentage of total ground volume that is filled with grout, ranges from about 3 to 12 percent. Additional information on compaction grouting can be found in Graf (1992a) and Warner et al. (1992).

Jet Grouting (US Army Corps of Engineers 1999)

Jet grouting is a process in which a high-pressure water jet is used to erode the native soil and mix it or replace it with a stabilizer such cement or bentonite, as depicted in Figure 1.6. The grout-soil mixture forms high strength or low permeability columns, panels or sheets, depending on the orientation and rotation of the jets as they are withdrawn from the ground. Columns of up to about 1 m diameter are typical, although much larger columns are possible using special equipment. Jet grouting can be used in most soil types, although it works best in soils that are easily eroded, such as cohesionless soils. Cohesive soils, especially highly plastic clays, can be difficult to erode and can breakup in chunks. The return velocity of the drilling fluid is usually not large enough to remove chunks of clay, so the quality of the grout-soil mixture could be compromised and hydrofracturing could occur in highly plastic clays (ASCE, 1997). A drawback of jet grouting is that it is very expensive and that special equipment is required. However, one advantage is that treatment can be restricted to the specific layer requiring improvement. Another advantage is that the injection rods can be inclined, so it is useful for grouting under structures or existing facilities. Burke and Welsh (1991) and Xanthakos et al. (1994) can be consulted for additional information regarding jet grouting.

Soil Replacement (US Army Corps of Engineers 1999)

Soil replacement involves excavating the soil that needs to be improved and replacing it. The excavated soil can sometimes be recompacted to a satisfactory state or it may be treated with admixtures and then be replaced in a controlled manner. It can also be replaced with a different soil with more suitable properties for the proposed application. This treatment is limited to upper layers usually less than 2 m.

Admixture Stabilization (US Army Corps of Engineers 1999)

Admixture stabilization consists of mixing or injecting admixtures such as cement, lime, or lime-flyash into a soil to improve its properties. Admixtures can be used to increase the strength, decrease the permeability or improve the workability of a soil. Admixtures can fill voids, bind particles, or break down soil particles and form cement. The general process of admixture stabilization consists of (1) excavating and breaking up the soil, (2) adding the stabilizer and water, if necessary, (3) mixing thoroughly, and (4) compacting the soil and allowing it to cure. Admixture stabilization is discussed in detail in Hausmann (1990).

MSE Walls (Reinforced Earth)

The MSE structure is a composite coherent gravity mass consisting of a wall panel facing element, and horizontal reinforcing strips as shown in Figure 1.7. The reinforcement may be either metallic or a geosynthetic, and serves to provide tensile strength to the soil. The system is used in conjunction with precast concrete facing panels, and following the placement of the base course of facing panels, each additional layer of panels interlocks with the previous course. Reinforcing strips and backfill are then placed and compacted in successive layers in a similar manner similar to the placement of traditional earthen embankments. Because the transfer of tensile stress from the reinforcement to soil is frictional, strict specifications are set for the physical and chemical properties of the backfill, to ensure good overall performance of the system. These include upper and lower gradation limits of 100 per cent passing a 150 mm sieve and, with certain exceptions, a 15 per cent limit on the percentage passing the 0.075 mm (200 mesh sieve). In addition, limits are set on electrochemical properties including Chlorides, Sulphates, Resistivity and pH (acidity), for both above water and underwater applications.

Design guidelines are given in FHWA (1996a) Mechanically Stabilized Earth Walls and Reinforced Slopes: Design and Construction Guidelines Publ. No. FHWA-SA-96-071.

Reinforced Slopes

Reinforced slopes extend the concept of MSE walls to slopes as shown in Figure 1.8. Whereas MSE walls are nearly vertical, reinforced slopes are typically between 1H:1V to 1H:2.5V. Conceptually, for a uniform fill soil there is a limiting slope angle β lim to which an unreinforced slope may be safely built. For the case of a non-cohesive and dry material, the limit angle of the slope equals the friction angle of the soil: $\beta_{lim} = \phi$. A slope with a greater angle than the limiting slope angle is a steep slope; to build an embankment with a steep slope it is

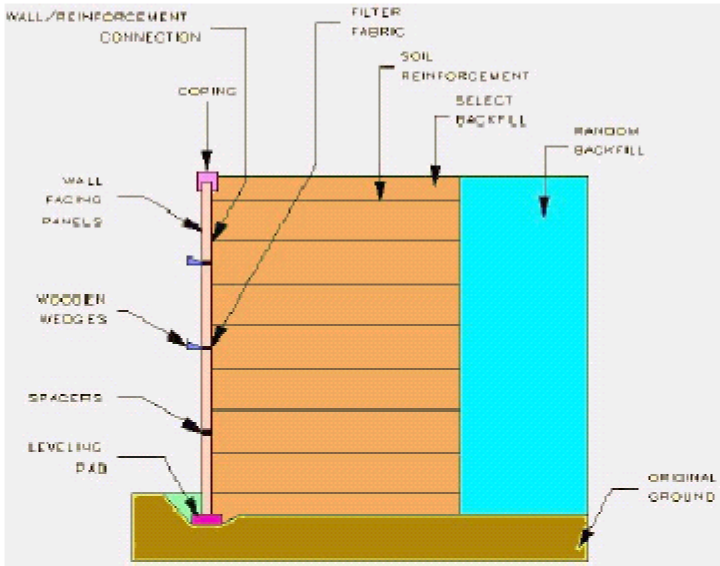


Figure 1.7 Schematic of MSE Wall (Passe, 2000)

necessary to provide some additional tensile forces to maintain equilibrium. The easiest method is to place horizontally some reinforcing layers in the slope so that the reinforcements can resist the horizontal forces, thus increasing the allowable shear stresses. Reinforcement is usually geogrid mats (sheets).

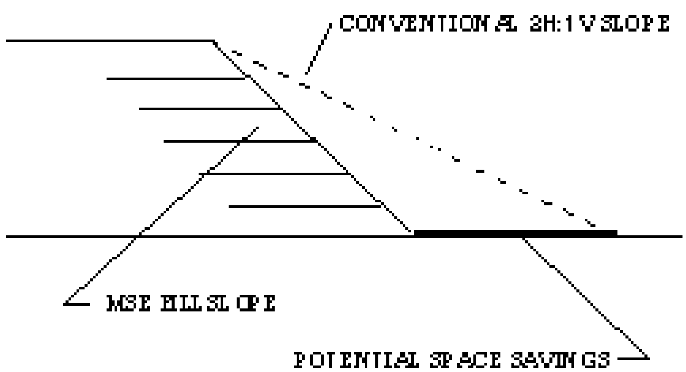


Figure 1.8 Illustration of Reinforced Slope

SOIL NAILING (US ARMY CORPS OF ENGINEERS 1999)

Soil nailing consists of a series of inclusions, usually a grout-filled hole about 6 inches in diameter with a 1 inch steel rebar, in the ground to be supported. By spacing the inclusions closely, a composite structural entity can be formed. The "nails" are usually reinforcing bars 20-30 mm in diameter that are grouted into predrilled holes or driven using a percussion drilling device at an angle of 10 to 15 degrees down from the horizontal. Drainage from the soil is provided with strip drains and the face of the excavation is protected with a shotcrete layer. The purpose of soil nailing is to improve the stability of slopes or to support slopes and excavations by intersecting potential failure planes. An example of soil nailing for excavation support is shown in Figure 1.9. There are two mechanisms involved in the stability of nailed soil structures (Mitchell and Christopher, 1990). Resisting tensile forces are generated in the nails in the active zone. These tensile forces must be transferred into the soil in the resisting zone through fiction or adhesion mobilized at the soil-nail interface. The second mechanism is the development of passive resistance against the face of the nail.

Soil nailing works best in dense granular soil and stiff, low plasticity silty clay soils. In stiff soils, the maximum facing displacement is about 0.3 percent. Current design procedures for soil nailed walls are included in FHWA (1996b).

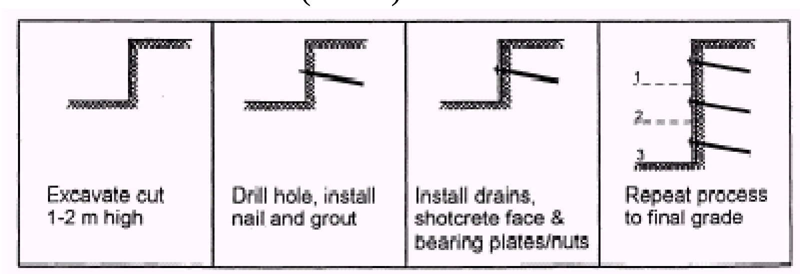


Figure 1.9 Soil Nailing for Excavation Support (after Walkinshaw and Chassie, 1994)

Stone Columns (Vibroreplacement) (US Army Corps of Engineers 1999)

Stone columns are installed using a process similar to Vibrocompaction, except that a gravel backfill is used, and they are usually installed in slightly cohesive soils or silty sands rather than clean sands. In the dry process, a cylindrical cavity is formed by the vibrator, that is filled from the bottom up with gravel or crushed rock. Compaction is by vibration and displacement during repeated 0.5± m withdrawals and insertions of the vibrator. Stone columns are usually about 1 m in diameter, depending on the soil conditions, equipment and construction procedures. They are usually installed in square or triangular grid patterns, but may also be used in clusters and rows to support footings and walls. Center-to-center column spacings of 1.5 to 3.5 m are typical. Figure 1.10 illustrates the construction method. For foundation applications, coverage should be extended beyond the perimeter of the structure to account for stress spread with depth. A drainage blanket of sand or gravel 0.3 m or more in thickness is usually placed over the top of the treatment area. This blanket also serves to distribute stresses from structures above. Additional details regarding stone columns are discussed in Mitchell (1981), Hausmam (1990), and Hayward Baker (1996).

Geogrids *(Tensar, 2002)*

Weak subgrades are a common problem in pavement construction. As the foundation for a pavement, subgrade failure will lead to rapid deterioration of the pavement structure. Traditionally, weak or pumping subgrades have been removed and replaced with imported fill or

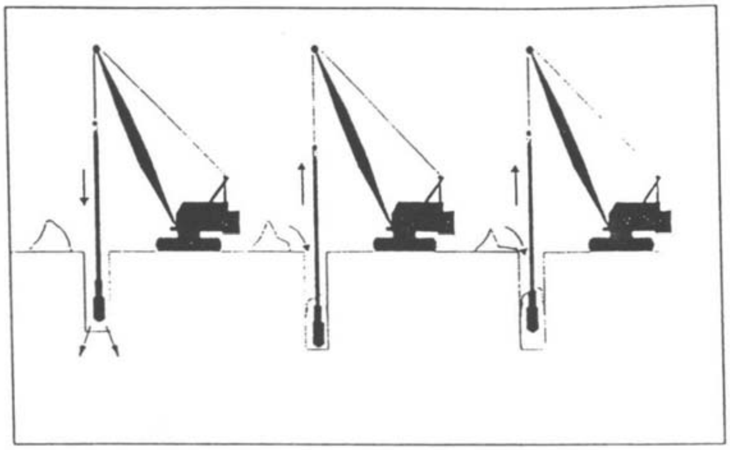


Figure 1.10 Illustration of Stone Column Construction (from FHWA, 1999)

stabilized chemically. Both options are expensive and time-consuming. The concept of geogrid reinforcement is to improve the performance of the existing subgrade by distributing loads over a wider area which reduces pumping and shear failure while maximizing the load bearing capacity of the subgrade as shown in Figure 1.11. Consequently, reinforcement can reduce or even eliminate the need for undercutting, for removing weak or contaminated soils, and for importing expensive, select fill. The results are faster construction and lower costs. Geogrids can reduce the thickness of the fill layer by as much as 50% while achieving the needed support.

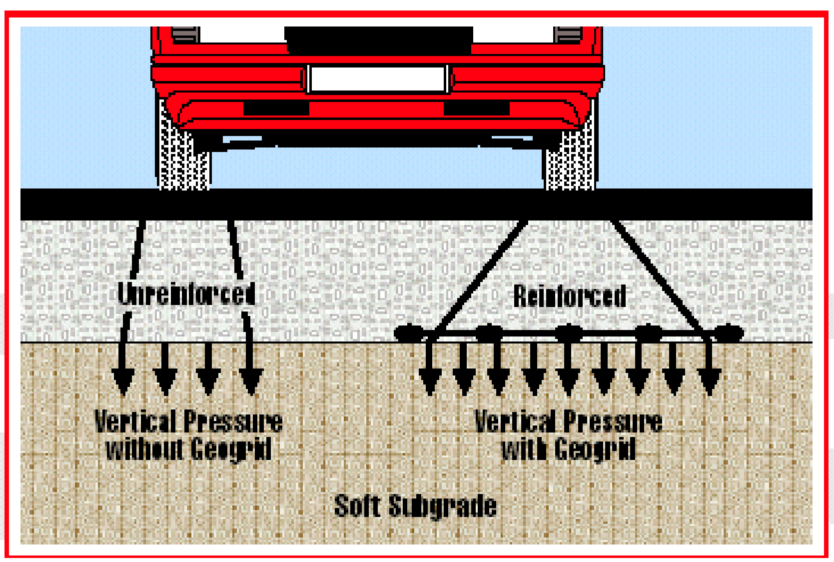


Figure 1.11 Illustration of Base Course Geogrid Reinforcement (from Tensar, 2002)

Lightweight Fills

Since the compacted unit weight of most soils ranges from 1.8 to 2.0 kN/m^3, in some highway applications it may be desirable to use lightweight materials to minimize settlements, or stability issues. Lightweight fill materials consist of wood fibers, flyash, slags, geofoam (expanded polystyrene), and shredded tires. Offsetting the advantage of reducing the applied stresses, if the observation that most light weight fill materials are quite compressible (wood fibers, and shredded tires).

Deep Soil Mixing

In the deep soil mixing technique, admixtures are injected into the soil at the treatment depth and mixed thoroughly using large-diameter single- or multiple-axis augers to form columns or panels of treated material. The mix-in-place columns can be up to 1 m or more in diameter. The treatment modifies the engineering properties of the soil by increasing strength, decreasing compressibility, and decreasing permeability. Typical admixtures are cement and lime, but slag or other additives can also be used. The mix-in-place columns can be used alone, in groups to form piers, in lines to form walls, or in patterns to form cells. The process can be used to form soil-cement or soil-bentonite cutoff walls in coarse-grained soils, and to construct excavation support walls. A detailed discussion of deep mixing is presented in ASCE (1997).

REFERENCES

ASCE Soil Improvement and Geosynthetics Committee, Ground Improvement Subcommittee (1997).

Burke, G.K. and Welsh J.P. (1991). "Jet Grout Uses for Soil Improvement." ***Geotechnical Engineering Congress,*** Geotechnical Special Publication No. 27, ASCE, Vol. 1, pp. 334-345.

ETL 1110-1-185 (1 Feb 99) Guidelines on Ground Improvement for Structures and Embankments, U.S. Army Corps of Engineers, Washington, D.C.

FHWA (1998). ***Ground Improvement Technical Summaries, Final Draft,*** FHWA Demonstration Project No. 116, Contract No. DTFH61 -93-R-00 145, Elias, V., Welsh, J., Warren J. and Lukas R.

FHWA (1996a). Publication Nos. 96-019, 530 p. ***Drilled and Grouted Micropiles: State-of-Practice Review,*** Final Report FHWA-RD-96-016, FHWA-RD-96-017, FHWA-RD-96-018, FHWA-RD

FHWA (1996b). ***Drilled and Grouted Micropiles: State-of-Practice Review,*** Final Report. FHWA-RD-96-016, FHWA-RD-96-017, FHWA-RD-96-018, FHWA-RD ***Manual for Design and Construction Monitoring of Soil Naii Wa[[s.***

FHWA (1986a). ***Dynamic Compaction for*** Highway ***Construction, Volume 1: Design and Construction Guidelines.*** U.S. Department of Transportation Federal Highway Administration Research, Development and Technology, Report No. FHWA/RD-86/133, Lukas, R.G.

FHWA (1986c). ***Prefabricated Vertical Drains.*** U.S. Department of Transportation Federal Highway Administration Research Development and Technology Volume 1, Engineering Guidelines. Report No. FHWA/RD-86/168.

FHWA (1983). ***Design and Construction of Stone Columns.*** Volume 1, Report No. 1,

FHWA/RD 83/026, Federal Highway Administration Barksdale, R.D. and Bachus, R.C., 210 P.

FHWA Demonstration Project 103, Publication No. FwA-SAO-96-069, Golder Associates, Inc.

Graf E.D. (1992). "Compaction Grout, 1992." ***Grouting, Soil Improvement and Geosynthetics, Geotechnical Special Publication No. 30,*** ASCE, Vol. 1, pp. 275-287.

Hausmann, M.R. (1990). ***Engineering Principles of Ground Modification,*** McGraw-Hill, USA 632 p.

Hayward Baker (1996). ***Ground Modification Seminar Notes.*** Hayward Baker, Odenton MD.

Karol, R.H. (1990). ***Chemical Grouting,*** Marcel Dekker, Inc., New York NY, 465 p.

Mitchell, J.K. (1981). "Soil Improvement: State-of-the-Art." State-of-the-Art Report, Session 12, 10th ***International Conference on Soil Mechanics and Foundation Engineering,*** Stockholm Sweden.

Mitchell, J.K. and Christopher, B.R. (1990). "North American Practice in Reinforced Soil Systems." ***Proceedings of a Conference, Design and Performance of Earth Retaining Structures,*** June 1990, ASCE Geotechnical Special Publication No. 25, pp. 322-246.

Passe, Paul. (2000) ***Mechanically Stabilized Earth Wall Inspector's Handbook,*** Florida Department of Transportation.

"Soil Improvement and Geosynthetics Committee Report," ***Ground Improvement, Ground Reinforcement and Ground Treatment, Developments*** 1987-1997, Geotechnical Special Publication No. 69, Proceedings of the sessions sponsored by the Committee on Soil Improvement and Geosynthetics of the Geo-Institute of ASCE in conjunction with Geo-Logan '97, Logan, Utah July 1997, V.R. Schaefer, cd., ASCE, pp. 1-371.

Tensar (2002) www.tensarcorp.com

Warner, J., Schmidt, N., Reed, J. Shepardson D., Lamb, R. and Wong, S. (1992). "Recent Advances in Compaction Grouting." ***Grouting, Soil Improvement and Geosynthetics, Geotechnical Special Publication No. 30,*** ASCE, Vol. 1, pp. 252-264.

Xanthakos , P.P., Abramson, L.W. and Bruce, D.A. (1994). Ground Control and Improvement, John Wiley & Sons, New York NY, 910 p.

Chapter 2

COMPACTION

INTRODUCTION

Densification of soils by mechanical compaction is the most common and oldest ground modification technique. Essentially, *compaction* is the reduction in the volume of voids by mechanical means; as contrasted with *consolidation* which also is a reduction in the volume of voids, but by long-term static loading. Hence compaction is the reduction of air voids due to imparting of mechanical energy, time is not a factor, the water content "w" remains constant, and the percent saturation increases.

Likewise, compaction using mechanical equipment is restricted the near surface layers.

To answer the question: why compact? Other than chemical stabilization, compaction is the only method for altering or influencing the engineering properties of soils. The major properties of interest by compacting are:

1. Strength increase
2. Permeability reduction
3. Volumetric stability, in that compressibility is reduced and the soil is stiffer
4. Liquefaction potential is reduced
5. Shrink-swell volumetric changes are mitigated.

To answer the question: how do we obtain the desired properties? The assumption is that these beneficial engineering properties are correlated with density (unit weight) and water content, and if we compact to those specified densities (unit weights) and water content conditions we will obtain the desired property. However, this assumption is clouded in that the specified density and water content are derived from laboratory tests, but field equipment and field conditions do not replicate laboratory conditions. Seldom are field tests performed to verify that the desired properties were obtained. Consequently, engineering reason is required for successful compaction projects.

Today, compaction projects usually consist of:

1. Specifying placement conditions of density and water content based upon laboratory compaction tests, and lift thicknesses.
2. Selection of appropriate field equipment (rollers, tampers) and operation (number of passes, pattern of rolling).
3. Field preparation of soil to proper water content by drying or moistening.
4. Evaluation via field density and moisture content measurements.

LABORATORY PROCEDURES

The objective of the laboratory tests is to provide guidance for field compaction; specifically the placement densities achievable and corresponding water content required to achieve that density. The most common laboratory test is the Proctor (1930) standard compaction test, whereby a steel rammer is repetitively dropped to compact via *impact* blows loose soil inside a steel cylindrical mold. The water content is changed from sample to sample to develop a relationship between dry density and water content. Other less common laboratory compaction methods include (1) *kneading* compaction (Harvard miniature), in which a spring loaded plunger is used to compact the loose soil inside the mold, (2) *static* compaction, whereby a static load is used to compress the loose soil to a prescribed density, and (3) *vibratory* compaction, in which, cohesionless soils are vibrated in a mold placed upon a vibratory table beneath a 2 psi surcharge.

Impact (Proctor) Compaction

The test was developed by R. R. Proctor in the early 1930's while working on compacted earth dams in California. He established that the dry density of compacted soil is dependent upon: (1) water content, w, (2) compactive effort, and (3) soil characteristics (grain size, mineralogy, gradation). The equipment and procedures are similar as those proposed by R. R. Proctor (*Engineering News Record*—September 7, 1933) with this one major exception: his rammer blows were applied as "12 inch firm strokes" instead of free fall, producing variable compactive effort depending on the operator, but probably in the range 15,000 to 25,000 ft-lb/ft^3 (700 to 1,200 kN-m/m^3). The procedure is described as ASTM standard D-698 for *standard* effort, and D-1557 for *modified* effort. Figure 2.1 illustrates the (1) mold, (2) falling weight (sleeve-type) rammer, (3) water bottle for adding moisture, (4) tare cans for water content samples, and (5) sieve for removing oversized particles.

Figure 2.1 Proctor Compaction Equipment

Three alternative methods are used for the standard proctor compaction test (ASTM 698) depending upon whether a 4-in. - diameter or 6-in. - diameter mold is used and upon the maximum particle size.

1. *Method A* uses: a 4-in. (101.6-mm) diameter mold, for material passing the No. 4 (4.75-mm) sieve and may be used if 20 % or less by mass of the material is retained on the No. 4 (4.75-mm) sieve. Compactive effort is 25 blows per layer.

2. *Method B* also uses a 4-in. - diameter mold, but is for material passing 3/8-in. (9.5-mm) sieve, and shall be used if more than 20 % by mass of the material is retained on the No. 4 (4.75-mm) sieve and 20 % or less by mass of the material is retained on the 3/8-in. (9.5-mm) sieve. Compactive effort is 25 blows per layer.

3. *Method C* uses a 6-in. (152.4-mm) diameter mold, and is for material passing ¾ -inch (19.0-mm) sieve. It shall be used if more than 20 % by mass of the material is retained on the 3/8-in. (9.5-mm) sieve and less than 30 % by mass of the material is retained on the ¾ -in. (19.0-mm) sieve. Also the compactive effort is increased from 25 blows per layer to 56 blows per layer.

4. If the test specimen contains more than 5 % by mass of oversize fraction (coarse fraction) and the material will not be included in the test, corrections must be made to the unit mass and water content of the specimen or to the appropriate field in place density test specimen using Practice D 4718.

5. This test method will generally produce a well defined maximum dry unit weight for non-free draining soils. If this test method is used for free draining soils the maximum unit weight may not be well defined, and can be less than obtained using Test *Methods D* 4253.

In summary, the ASTM 698 is a laboratory compaction methods used to determine the relationship between water content and dry unit weight of soils (compaction curve) compacted in a 4 or 6-in. (101.6 or 152.4-mm) diameter mold with a 5.5-lbf (24.4-N) rammer dropped from a height of 12 in. (305 mm) producing a compactive effort of 12,400 ft-lbf/ft^3 (600 kN-m/m^3). These test methods are restricted to soils (materials) that have 30 % or less by mass of particles retained on the 34-inch (19.0-mm) sieve.

Compactive Effort

The compactive effort for the Standard compaction test is:

Method A or B

25 drops, 5.5 lb hammer, 12-inch drop, on each of 3 layers, in a 4-inch diameter mold (1/30th cu.ft.).

This corresponds to a compactive effort of:

(5.5 lbs)(1-ft. drop)(25 blows/layer) (3 layers)/(1/30th ft^3) = 12,375 ft lbs/cu.ft. = equivalent to a light roller.

For Method C, the number of drops is increased to 56 drops/layer to accommodate using a larger mold – 6-in.- diameter (0..075 ft^3), instead of 4-in.- diameter; or (5.5 lbs) (1-ft drop)(56 blows/layer)(3 layers)/(0.075 ft^3) = 12,319 ft-lbs/ft^3.

Modified Proctor Compaction Test (ASTM D-1557)

The modified compaction was developed by the U.S. Army Corps of Engineers during World War II to duplicate the compaction requirements for heavy aircraft on airfields. Three alternative methods are used in ASTM D-1557, and are for soils having 30 % or less by mass of their particles retained on the 34-in. (19.0-mm) sieve. Methods A, B, and C have the same gradation requirements as for ASTM D-689. The modified compactive effort is an increase in the rammer weight, from 5.5 lbs to 10 lbs, the drop height is increased from 12-inches to 18-inches, and the number of layers is increased from 3 to 5. Consequently, for Methods A and B the modified compactive effort is:

25 drops, 10 lb hammer,18-inch drop, on each of 5 layers, in a 4-inch diameter mold (1/30th cu.ft.).

This corresponds to a compactive effort of:

$$(10 \text{ lbs})(1.5\text{-ft.drop})(25 \text{ blows/layer}) (5 \text{ layers})/(1/30^{th} \text{ ft}^3) = 56{,}250 \text{ ft. lbs/cu.ft.}$$

For Method C, the compactive effort will be:

$$(10 \text{ lbs})(1.5\text{-ft. drop})(56 \text{ blows/layer}) (5 \text{ layers})/(0.075 \text{ ft}^3) = 56{,}000 \text{ ft. lbs/cu.ft.}$$

Table 2.1 summarizes the standard and modified Proctor tests.

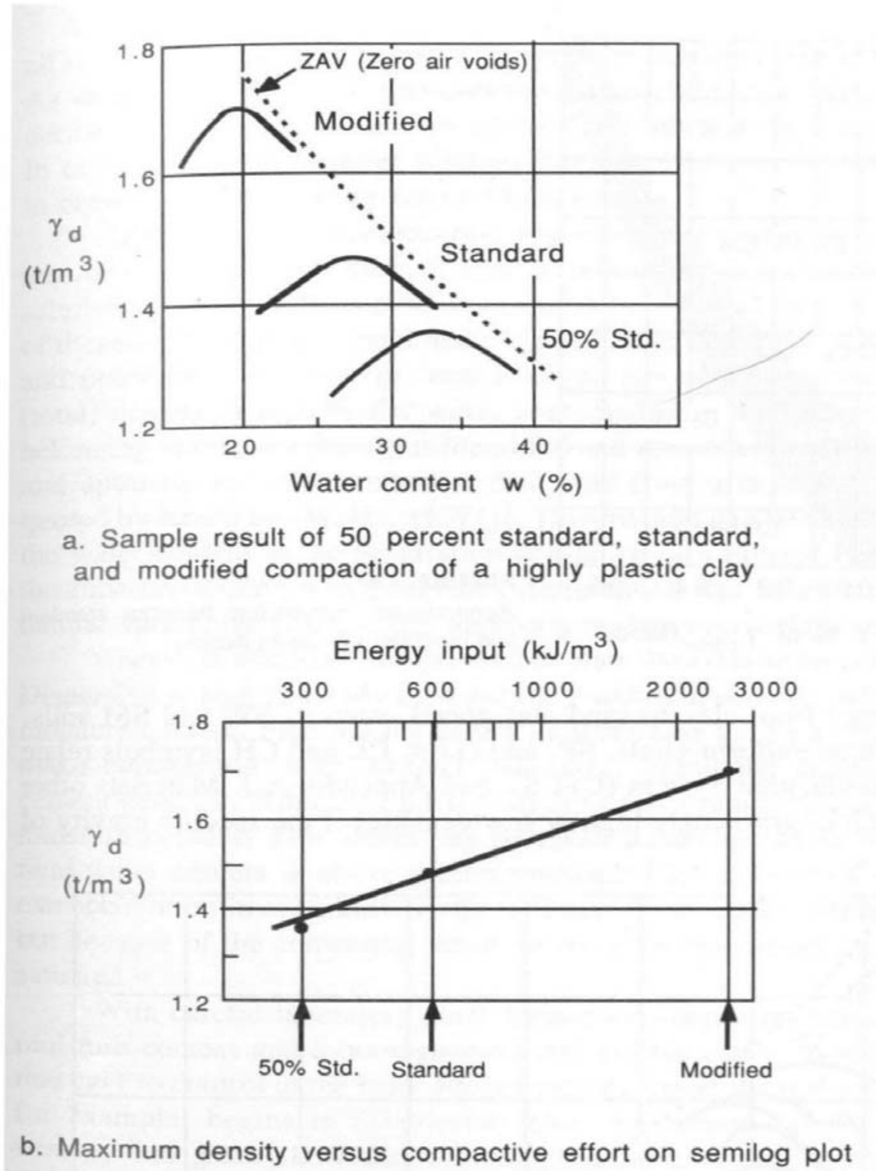
Table 2.1 Summary of Standard and Modified Proctor Tests

Text	**Mold**	**Hamme r Wt.**	**No. of Layers**	**Ht. of Hammer Drop (m)**	**No. of Drops per Layer**	**Compactive Energy per Unit Volume (ft-lbs / ft^3)**
Standard Proctor	4.6 × 4 in. dia.	5.5	3	12	25	12400
Standard Proctor	5 × 6 in. dia.	5.5	3	12	55	12400
Modified Proctor	4.6 × 4 in. dia.	10	5	15	25	56000
Modified Proctor	5 × 6 in. dia.	10	5	15	55	56000
15 Blow Proctor	4.6 × 4 in. dia.	5.5	3	12	15	7400
15 Blow Proctor	5 × 6 in. dia.	5.5	3	12	35	7400

The compaction test results are plotted as dry (unit weight) density versus water content as illustrated in Figure 2.2 and 2.3.

The dry density (γ_{dry}) is calculated from the total (or wet density, γ_{wet}) as:

$$\gamma_{dry} = \frac{\gamma_{wet}}{1+w}$$



a. Sample result of 50 percent standard, standard, and modified compaction of a highly plastic clay

b. Maximum density versus compactive effort on semilog plot

Figure 2.2 Compaction Test Results – Dry Unit Weight vs. Water Content

(from Hausmann, 1990)

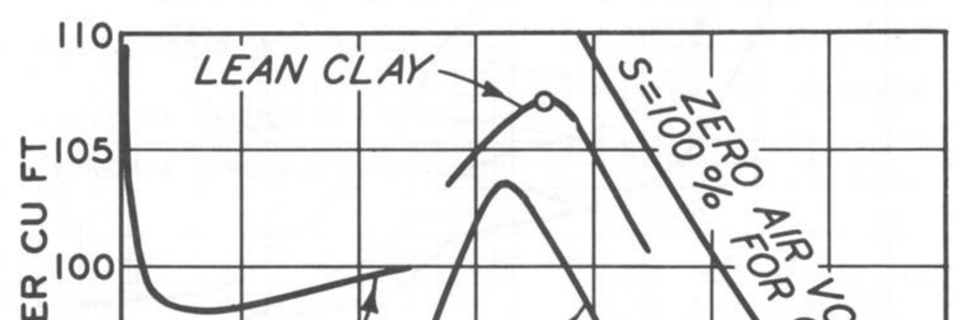


Figure 2.3 Illustration of Dry Density vs. Water Content Compaction Curves

(from US Army-Corps of Engineers EM 1110-2-1906, 1970)

where w represents the water content. The density is usually expressed as mass units [metric tons/m^3] or force units [kN/m^3, lbs/ft^3]. Some engineers use ρ instead of γ if mass units are used (Hausmann, 1990). The zero-air-voids (ZAV) curve represents 100% saturation and serves as a boundary for which data points cannot cross; i.e., cannot have more than 100% saturation. The ZAV curve can be calculated as:

$$\gamma_{dry} = \frac{G_s \, \gamma_w}{1 + w G_s / S}$$

where G_s = specific gravity, w = water content, and S = saturation (decimal)

The moisture - density relationship depends on the energy input. The higher the effort, the higher the maximum density and the lower the optimum moisture content. The "line of optimums" is approximately parallel to the "zero air voids" curve (Figure 2.4).

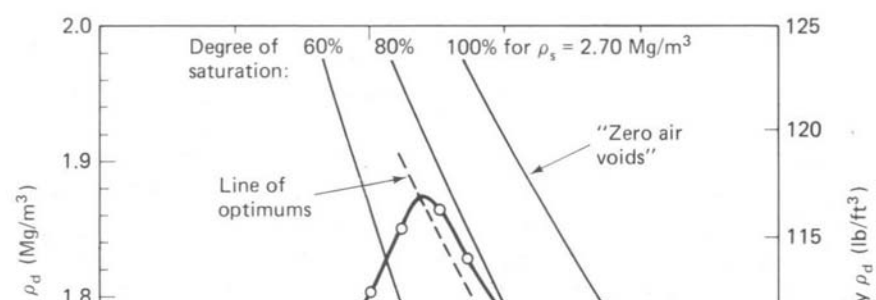


Figure 2.4 Effect of Compaction Energy on Compaction Curves

Kneading Compaction

The Harvard Miniature Compaction Test was developed by S. Wilson while at Harvard University in 1953. The compactor is a spring loaded tamper and uses a mold with a volume of 1/454 cu.ft.(Figure 2.5) Hence the weight of the compacted sample in gms is equivalent to the unit weight in pcf. The spring loaded tamper uses 10, 15, and 20 lb. springs. The tamping action simulates the kneading compaction of a sheepsfoot roller. Compaction is in four equal lifts, each receiving 25 blows from the spring-loaded tamper in such a manner as to uniformly cover the entire surface of the material.

Vibratory Compaction (ASTM D 4253 – 00, Standard Test Methods for Maximum Index Density and Unit Weight of Soils Using a Vibratory Table)

Inasmuch as cohesionless compact best by vibration, the vibratory compaction test method covers the determination of the maximum–index dry density/unit weight of cohesionless, free-draining soils using a vertically vibrating table. The test consists of vibrating for 12 minutes using a vibrating table a specimen subjected to a surcharge of 2 psi. The results are expressed as relative density:

$$D_R = \frac{e_{max} - e}{e_{max} - e_{min}} \times 100 \, (\%)$$

Figure 2.5 Harvard Miniature Compaction Apparatus

where: e_{max} = void ratio when gently pouring soil into mold

e_{min} = void ratio after vibration

The test method designed to simulate field vibratory compaction of cohesionless soils.

Applicability of Laboratory – Field Compaction

Table 2.2 compares applicability of laboratory test to field simulation.

Table 2.2 Comparison of Laboratory and Field Compaction Applications

(from Univ. of Washington, Department of Civil Engineering Soil Lab. Notes)

Method	Lab Simulation	Field Technique
Impact	Standard compaction test	Nothing comparable
Kneading	Harvard miniature apparatus; Hveem method	Sheepsfoot roller, wobble wheel; Rubber-tired roller
Vibration	Vibratory table	Vibratory rollers and compactors
Static (or dynamic) compression	Compression machines	Smooth wheel rollers

Field Compaction Equipment

When soil is used as a construction material (earth dams, levees, dikes, embankments, road bases, etc.) it is usually placed in layers (lifts) and each layer compacted before the next placed - compacted earth fill. Compaction is the densification of soils by the application of mechanical energy.

Compaction in the field is performed with rollers. There are five types; smooth wheel or drum, pneumatic or rubber-tired, sheepsfoot, mesh or grid, and vibratory.

The ***smooth wheel, or drum, roller*** (Figure 2.6) produces 100% coverage under the wheel, with ground contact pressures up to 55 psi. Can be used on sandy or clayey soils. Commonly used for proof rolling subgrades, finishing operation on fills and compacting asphalt pavements. Not suitable for producing high unit weights when used on thick lifts.

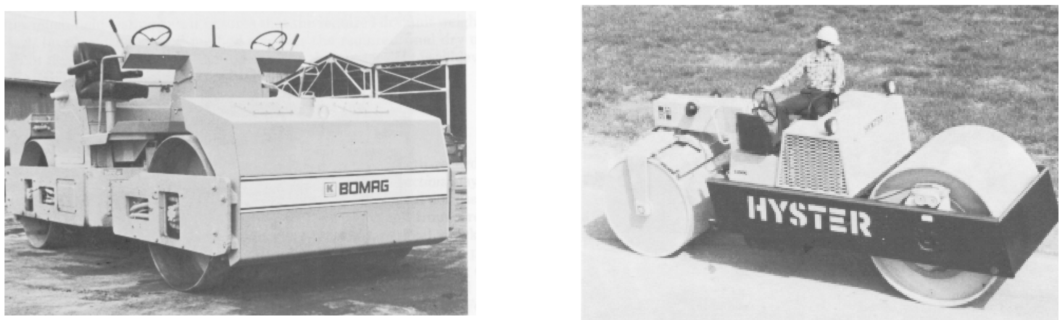


Figure 2.6 Smooth Steel Wheel Rollers

The ***pneumatic, or rubber-tired, rollers*** (Figure 2.7) are heavily loaded wagons with several rows of closely spaced tires (four to six in a row). May be towed or self-propelled. They have about 80% coverage and tire pressures up to 100 psi. Can be used on both sandy and clayey soils. Compaction is achieved by a combination of pressure and kneading action.

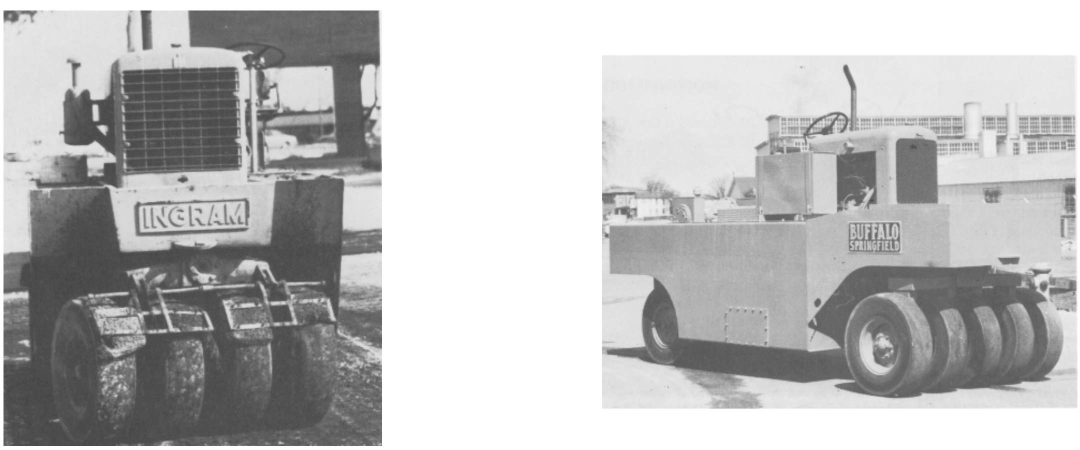


Figure 2.7 Rubber-Tired Rollers

The ***sheepsfoot roller*** (Figure 2.8) has many round or rectangular protrusions or “feet” attached to a steel drum (diameter 40 - 60 inches). As many as 112 to 144 feet, usually in rows of 4 are attached to the drum. The “foot” length varies from 5 to 9 –inches, with a projected area ranging from 4 to 13 in^2. Also, the shape of the feet varies; they may be club, taper, or wedge shaped. Because of the 8% to 12% coverage, very high pressures are possible, 200 to 1000 psi, depending on the drum size and whether it is filled with water. Sheepsfoot rollers are usually towed in tandem by crawler tractors or are self-propelled. The sheepsfoot roller starts compacting the soil below the bottom of the foot and works its way up the lift as the number of passes increases - it “walks out” of the fill. It is best suited for cohesive soils.

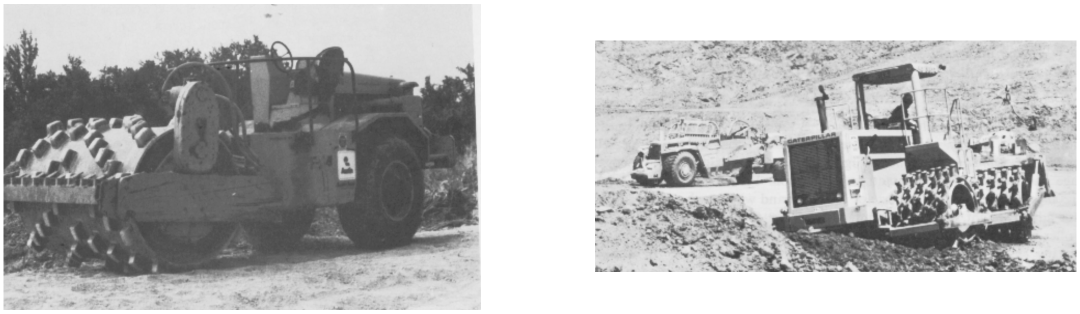


Figure 2.8 Sheepsfoot Rollers

The ***mesh, or grid, roller*** (Figure 2.9) has about 50% coverage and pressures from 200 to 900 psi. it is used to compact rocky soils, gravels and sands. With high towing speed, the material is vibrated, crushed and impacted.

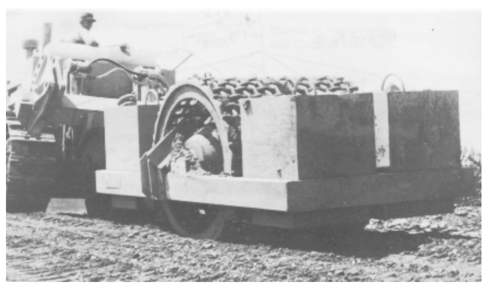


Figure 2.9 Grid Roller

Vibratory rollers (Figure 2.10) are very efficient in compacting granular soils, especially clean sands and gravels. They contain some kind of vibrating unit that imparts an up and down vibration to the roller - 1500 to 2000 cycles per minute.

Figure 2.10 Vibratory Roller

Table 2.3 (from Hausmann, 1990) summarizes the soil type compatibility with compaction equipment.

Table 2.3 Selection of Compaction Equipment

Equipment	Most suitable soils	Typical application	Least suitable soils
Smooth wheeled rollers,	Well graded sand-	Running surface,	Uniform sands

static or vibrating	gravel, crushed rock, asphalt	base courses, subgrades	
Rubber tired rollers	Coarse grained soils with some fines	Pavement subgrade	Coarse uniform soils and rocks
Grid rollers	Weathered rock, well graded coarse soils	Subgrade, subbase	Clays, silty clays, uniform materials
Sheepsfoot rollers, static	Fine grained soils with > 20% fines	Dams, embankments, subgrades	Coarse soils, soils with cobbles, stones
Sheepsfoot rollers, vibratory	as above, but also sand-gravel mixes	Subgrade layers	
Vibrating plates	Coarse soils, 4 to 8% fines	Small patches	clays and silts
Tampers, rammers	All types	Difficult access areas	
Impact rollers	Most saturated and moist soils		Dry, sands and gravels

Considerations: (1) *Lab test:* compactive effort, mold size, hammer, type (static, impact, vibratory, gyratory), (2) *Field compaction:* equipment, width of strip, lift depth, speed of travel, number of passes, maneuverability, (3) *Specs & field control:* field measurements, sand cone, nuclear, water balloon, calibration.

Meehan and Isrankura (1967) _Canadian Geotechnical Journal, Vol. IV, No. 3. Printed in Canada_ report on the uselessness of elephants *Elephas maximus*, the Asian elephant, in compacting fill. A 9-m square test section of a low plasticity sandy clay was divided into 9 square 20 cm thick sections at varying water contents. Compactive efforts were varied by using two elephants, a cow and a bull, and varying the number of passes. At the completion of the test, field density determinations (sand cone method) were made in each of the 9 squares.

Field densities and moisture contents are plotted alone with the Proctor density-moisture content curve in Figure 2.11. Numbers adjacent to the plotted points designate the number of elephant passes. It is clear from the test results that this method of compaction fails to produce adequate density. This is explained as follows:

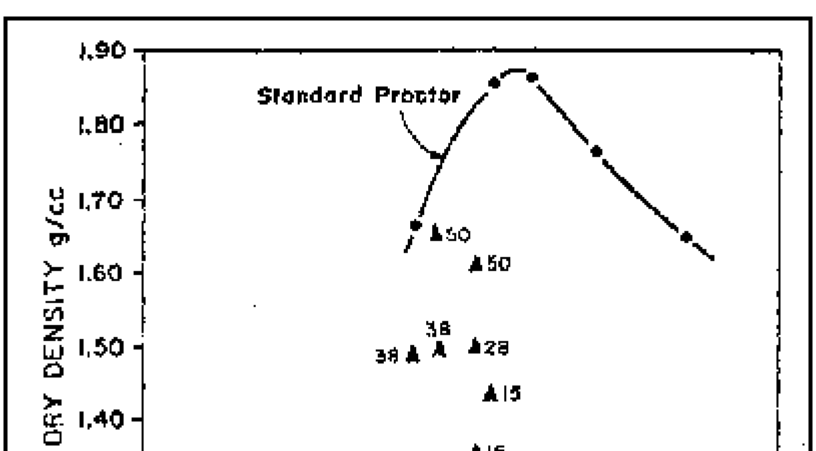


Figure 2.11 Comparison of Densities Achieved by Elephants

(from Meehan and Isrankura, 1967)

1. The weight of the animal is supported by three legs, even as it walks. The area of the imprint of each foot is typically 175 sq. cm, so that its weight is distributed over an area of about 525 sq. cm. Assuming a total weight of animal of 2000 kilograms, the applied pressure is 3.8 kg/sq. cm (54 lb./sq. in.). While this compares favorably with tire pressures of pneumatic rollers, the rate of coverage is much slower, i.e., a far greater number of passes is required for comparable coverage.
2. An elephant quickly learns to place its feet on precompacted areas and to avoid the softer uncompacted areas. The animal explores the terrain ahead with its trunk, a remarkable sensory organ, and generally will place its feet on or near previous imprints. Strenuous control by the handler is required to obtain uniform coverage.

It was concluded that elephants are inefficient compactors, even considering the local low cost (current rate of rental for one elephant and handler is $5 per 6-hour day).

Field Compaction Aspects

The objective of field compaction is to densify soils and thereby improve their engineering properties. However, these desired engineering improvements; i.e., increased strength, decreased permeability, etc. are measured in the field to ascertain compaction acceptance. Rather, construction compaction control is monitored by achieving a specified density and water content. It is assumed a prori that achieving a specified density and water content will result in achieving the desired engineering properties. Hence a strong dependence is placed upon the laboratory tests to establish this link between compaction characteristics and engineering properties. Fortunately, most engineering properties correlate well with compacted density and water content.

A field compaction project essentially follows these steps:

1. select borrow soil
2. haul to site and dump
3. spread into layers (few inches to 2 ft.)
4. alter moisture content .. lower by drying, raise by wetting

5. mix soil to make uniform, break lumps
6. compact the soil by rolling:
   (a) according to specified procedure - ***method specifications*** - type and weight of roller, the number of passes of that roller, as well as the lift thicknesses and moisture content range, are specified by the engineer (most common in large projects like an earth dam).
   (b) until specified properties are achieved - ***end-product specifications*** - a relative compaction is specified, the contractor can achieve this in any manner he chooses (common in highway and building foundations).

Factors Affecting Field Compaction

As in the case of laboratory compaction, factors affecting the compactive energy (effort) control the resulting density. Consequently, these factors are:

1. Weight of roller. Heavier rollers usually produce greater densities for fewer coverages (passes). However, if the bearing capacity of the soil conditions is exceeded, then heavier will not produce greater densities.
2. Number of coverages. A minimum of 4 to 8 passes is usually required for the efficient use rollers. A higher number of passes could lead to grain crushing and detrimental stratification among lifts. Therefore, minimizing the number of passes has both economical and technical advantages (Hausmann, 1990).
3. Contact pressure. Higher contact pressures by pneumatic tired rollers leads to deeper stress penetration and hence compaction of deeper lifts. Higher densities will result for greater tire pressures, provided the soil bearing capacity is not exceeded.
4. Lift thickness. The lift thickness is proportional to the pressure applied; i.e., higher pressures, thicker lifts. Forssblad (1977) suggests that a vertical stress of 50 to 100 kPa (7 to 14 psi) is sufficient for the vibratory compaction of cohesionless soils. Cohesive soils require greater stresses – 400 to 700 kPa (57 to 100 psi)
5. Water content. Water content is inversely proportional to bearing capacity, and thus is critical to field compaction efforts and equipment operation. A hierarchy exists for the number of passes for the density achievable at that water content; fewest to most: wet of optimum, optimum, and dry of optimum.

Field Compaction Control

The objectives of field compaction control tests are to determine inexpensively and effectively whether the appropriate density and water content have been achieved. Efficiency is paramount as acceptance or rejection of a compacted lift needs to be determined hastily so as not to impede the compaction schedule. Thus, density and water content determinations can be made *directly* (sand cone, and oven drying) or *indirectly* (nuclear probes).

Considering the measurements required for density are merely volume and weight, and those for water content are merely wet and dry weight, the traditional methods are to dig a small soil sample for the compacted lift, determine the hole volume, weigh the soil, dry, and reweigh. Determining the hole volume is the more difficult task of this operation.

Sand Cone Method (ASTM D 1556 – 00, Standard Test Method for Density and Unit Weight of Soil in Place by the Sand-Cone) – This method of determining hole volume relies upon filling the hole with a uniform sand, whose pour unit weight can be determined by calibration tests. The sand cone method is illustrated in Figure 2.12.

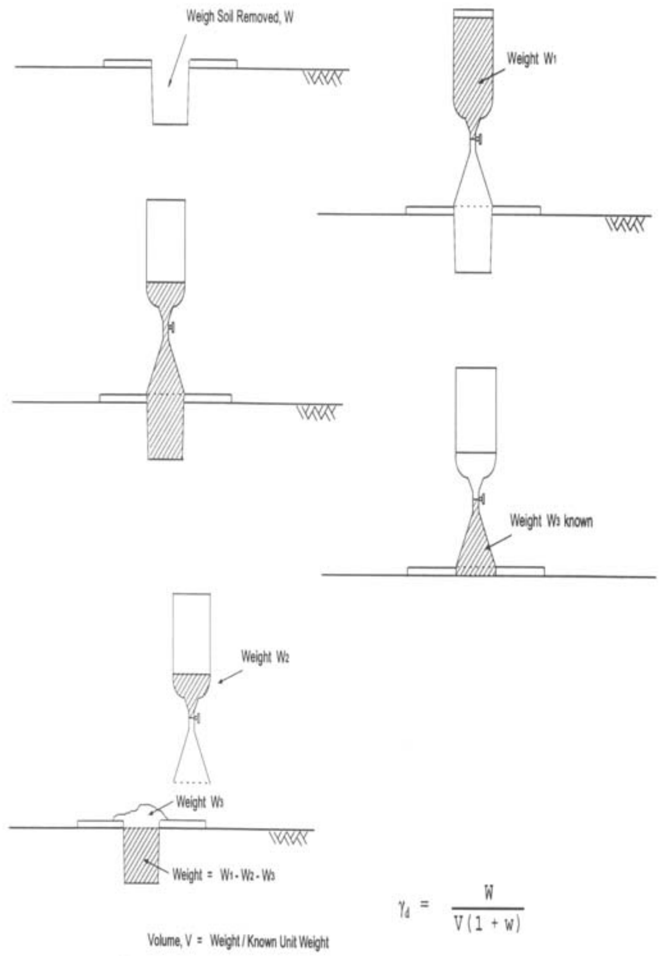


Figure 2.12 Illustration of Sand Cone Method

$$\gamma_d = \frac{W}{V(1 + w)}$$

Other less common methods include; (a) D2167-94 Standard Test Method for Density and Unit Weight of Soil in Place by the Rubber Balloon Method, and (b) D2937-00 Standard Test Method for Density of Soil in Place by the Drive-Cylinder Method. The rubber balloon method measures the hole volume by inflating a water filled rubber balloon until it completely fills the hole and measuring the volume of water; i.e., volume of hole as shown in Figure 2.13. The drive-cylinder method is restricted to cohesive soils whereby a cylindrical steel core cutter is driven into the lift and a known volume soil sample (plug) extracted.

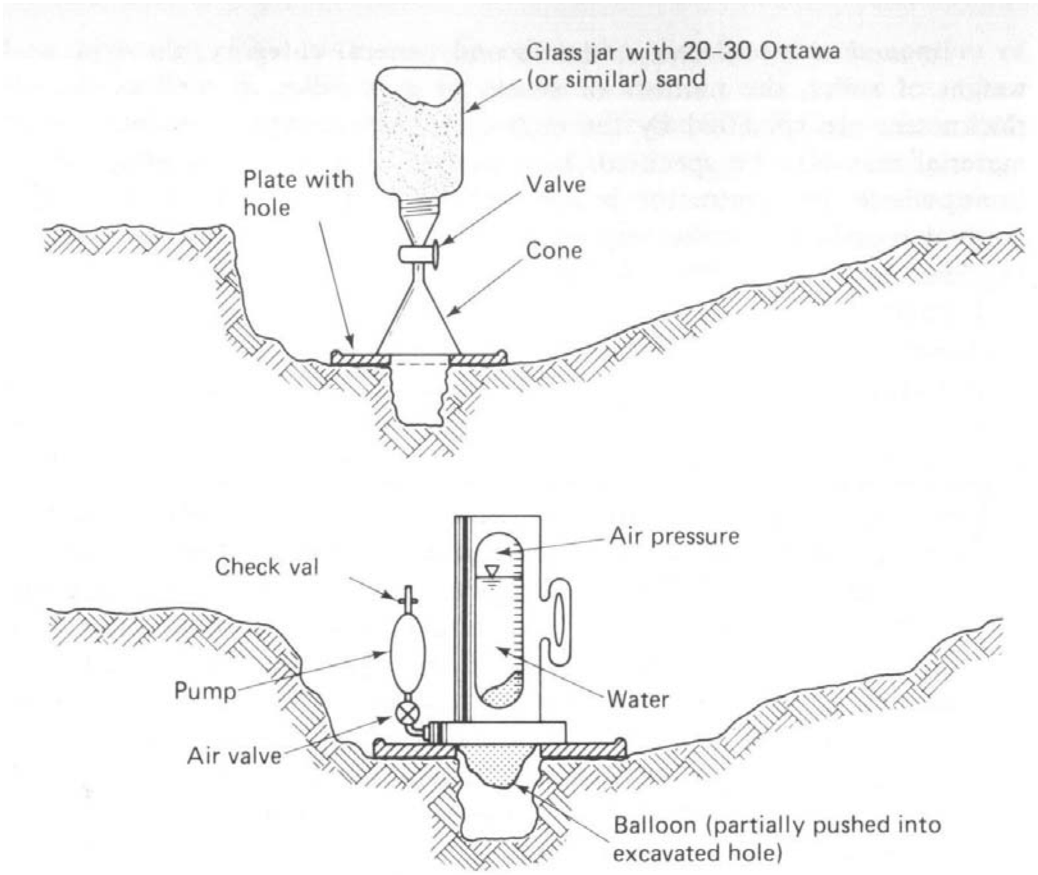


Figure 2.13 Rubber Balloon Method

Field Moisture Content

The laboratory method for determining a soil's moisture content is described in ASTM D 2216. Water is evaporated from the moist soil by drying in an oven (110 ± 5°C) until the soil mass is constant. This may require 12 hours or longer (often weighings are made after 24 hours). In the field, during compaction inspection, a more rapid determination is required.

Microwave Oven Method ASTM D 4643

A moist soil specimen is placed in a suitable container and its mass determined, M_1 . It is then placed in a microwave oven, subjected to an interval of drying, removed from the oven and its new mass determined. This procedure is repeated until the mass becomes nearly constant, M_2. The mass of water is $M_1 - M_2$. The mass of solids M_s is equal to M_2 - the mass of the container.

$$\text{Moisture or Water Content } w = (M_1 - M_2) * 100/M_s \ \%$$

A Computer Controlled Microwave Oven System (CCMOS) has been developed at WES (US Army Corps of Engineers, 1995) and demonstrated to be an acceptable and useful piece of equipment for rapid determination of water content for compaction control. The principal of operation of the system is that water content specimens are weighed continuously while being heated by microwave energy; a small computer monitors change in water content with time and terminates drying when all "free" water has been removed. CCMOS is essentially automatic; after the operator has placed a specimen in the oven system, the controlling computer performs all required tasks (including calculations) through software, and returns the final water content with no additional input required from the operator. A water content test in the CCMOS typically requires 10 to 15 min.

Field tests have shown that CCMOS produces water contents that are within 0.5 percent of the conventional oven water content. Special procedures must be used when drying materials which burst from internal steam pressure during microwave drying (which includes some gravel particles and shales) and highly organic material, which requires a special drying cycle. CCMOS will not produce correct water contents in soils with high gypsum content; therefore, no attempt should be made to use the system to dry such materials. (However, it must be noted that a special drying procedure is required to dry gypsum rich soils in the conventional constant temperature oven.) CCMOS and its operation and use are described by Gilbert (1990).

Speedy Moisture Tester (Figure 2.14)

Figure 2.14 Illustration of Speedy Moisture Tester

Moist soil is mixed with a calcium carbide reagent. The resulting gas pressure is correlated to moisture content. Equipment:- Speedy Moisture tester, tared scale, two steel balls, cleaning brush and cloth, scoop, calcium carbide reagent. The procedure is:

1. Weigh about 6g of soil and place in cap of the tester.
2. Place about 8g of calcium carbide in the larger chamber.
3. With the pressure vessel in a horizontal position lock the unit together.
4. Raise to a vertical position so the soil in the cap falls into the pressure vessel.

5. Shake vigorously. When the needle stops moving, read the dial while holding the instrument horizontally. This reading is the percent moisture by wet weight and must be converted to dry mass.

The pressure tester method for water content determination involves combining moist soil in a sealed chamber with calcium carbide (these react with water in the soil to release gas) and relating the resulting gas pressure to soil water content. Accuracy can be a problem when using this technique since soils and especially fine grained clays bind and hold water at different energy levels. Consequently, there is no assurance that calcium carbide will react correctly with bound or adsorbed water; calibration tests must be performed to correlate pressure tester water content with conventional oven water content. Special care should also be taken in using the pressure tester technique with organic soils, since accuracy is affected by the presence of organic matter in soils. The pressure tester technique is most effective and accurate on relatively dry soils (less than 20 percent water content) which readily disaggregate; the technique becomes cumbersome and possibly dangerous when testing excessively wet soils, as very high gas pressure may develop in the test chamber. The American Society for Testing and Materials (ASTM) has recently prepared a standard for the calcium carbide pressure tester method of water content determination (ASTM D 4944). The procedure states that uncertainty and sources of error in using the procedure arise from the fact that water entrapped in soil clods does not react correctly or completely with calcium carbide; additionally, some soils contain chemicals which react unpredictably with the reagent to give erroneous results. It is important to realize that when calcium carbide reacts with water, acetylene gas is released which is highly flammable to the point of being explosive; additionally, the gas is irritating to the skin and eyes. Therefore, appropriate safety measures must be exercised when using this procedure (US Army Corps of Engineers, 1995).

Indirect Field Measurements

Unit Weight and Moisture Content by Nuclear Methods ASTM D 2922

Unit weight measurements can be either (a) direct transmission (soil and aggregates), or (b) backscatter (concrete and hot asphalt). In the case of direct transmission, a steel rod is hammered into the soil to create a hole into which a radioactive source (eight millicurie cesium 137 source) is inserted. Gamma rays (photons) are emitted from a source into the soil. These collide with electrons in the soil materials. Some are scattered and some are absorbed. The quantity of photons reaching the Geiger counter detection device relates to the average soil unit weight between the probe and Geiger counter. The sensitivity for direct transmission is about ± 0.11 pcf in 120 pcf. Figure 2.15 illustrates the direct transmission mode (Troxler User's Manual 3430, 1990).

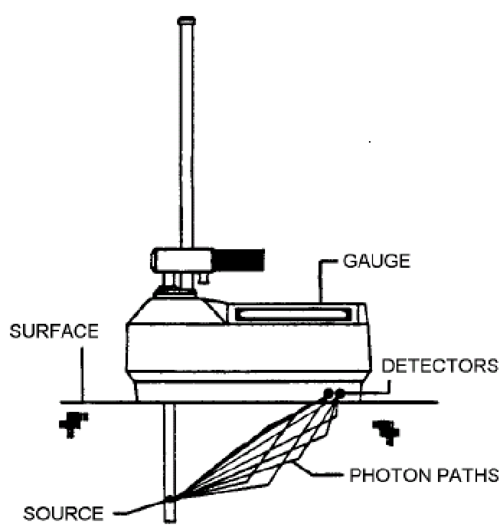


Figure 2.15 Nuclear Density Probe in Direct Transmission Mode

Moisture determinations are obtained using a source of high-velocity neutrons (forty millicurie americium-241:beryllium). When the fast neutrons collide with hydrogen, they are slowed down. The quantity of "slowed" neutrons are detected by a detector that is only sensitive to the "slowed" neutrons, and insensitive to the "fast" neutrons. As a result, the counts obtained are directly proportional to the amount of hydrogen (water) present in the material. The moisture content sensitivity is about ± 0.16 pcf in 15 pcf.

Neutrons lose energy primarily by colliding with chemically bound hydrogen present in the (soil-water) medium, and neutrons are absorbed by certain elements which may be present in the soil. Therefore, some of the factors that adversely affect water content measurement using this procedure may be more clearly visualized: (1) *All* chemically bound hydrogen causes neutron energy loss, including that in organic matter, adsorbed water, and structurally bound water as well as "free" water. Only free water should be included in a normal water content determination; the gauge cannot discriminate between hydrogen in free water and hydrogen in other sources. (2) Certain elements (such as iron, potassium, manganese, boron, and chlorine) are highly absorptive of neutrons. The presence of these elements in soils will cause erroneous water content determination using a nuclear gauge. Because of the possible presence of generally unknown quantities of organic matter, adsorbed water, structurally bound water, and highly absorptive elements, water content measured by the nuclear gauge must be frequently checked against that determined in the conventional oven to account for the factors which are known to influence nuclear gauge results, and to develop calibration curves. It should be noted here that recent research has shown that the calibration of nuclear gauges is highly nonlinear in determination of water content or soil density at water contents greater than about 40 percent, and steps should be taken to account for this nonlinearity.

In addition, nuclear gauges react with and are affected by other nuclear gauges in close proximity; therefore, a nuclear gauge should not be used within 30 to 40 ft of another nuclear gauge in use in the field. A major disadvantage of nuclear gauges is that specimen size is unknown and can never be established with certainty; the volume "probed" by a nuclear gauge is determined by water content, soil mineralogy, grain-size distribution, and geometry of the test configuration (for example, results determined in a narrow utility trench may be in considerable error relative to results obtained on a flat, obstruction-free soil surface). Additional disadvantages of nuclear methods for determining field densities and water contents are general lack of understanding of the method as well as factors affecting the results and, consequently, lack of confidence in the results; calibration curves must be developed and/or verified by field tests for each instrument; and although the proper use of nuclear gauges presents no health hazards, rigid safety regulations and documentation requirements must be met. For this last reason, field parties are sometimes reluctant to use nuclear equipment (US Army Corps of Engineers, 1995).

PROPERTIES OF COMPACTED COHESIVE SOILS

Lambe's (1958) Compaction Theory

One must recall that clay minerals are extremely small, and exhibit a net negative charge on the surface due to both isomorphic substitution, and imperfections in the crystal lattice. Water, being a dipolar molecule, while electrically neutral, displays a positive and negative end. Consequently, water is adsorbed on the clay surface via hydrogen bonding, where the positive end is held by the negatively charged clay. The attraction of water to the clay is quite strong at the clay surface and is essentially bonded to the clay mineral (1st layer). Moving outward from the clay surface, the clay negative charge strength decreases causing the water molecules to be less firmly attracted (2nd layer). These two water layers form the “double water layer”(DWL).

Figure 2.16 illustrates "typical" compaction curve. Point A on the “dry” side of optimum due to the lower water content, "w," will exhibit a depressed “Double Water Layer.” Consequently, the forces of attraction will dominate, and the clay particles tend to flocculate; i.e., edge to face. Because the double water layer is reduced comp active effort does little to change particle orientation. Conversely, at Point C on the “wet” side of optimum due to the higher water content, "w," forces of repulsion dominate, and the clay particles assume a dispersed (oriented) structure. However, due to the DWL thickness, compactive effort has little effect inasmuch as water is incompressible, and the clay platelets merely shift position relative to one another. Between Points A and C, lies Point B or “optimum water content.” At this condition what clay particle structure is between “flocculated” and “dispersed,” the DWL sufficient to allow movement of particles to the densest configuration, without being too thick to be incompressible.

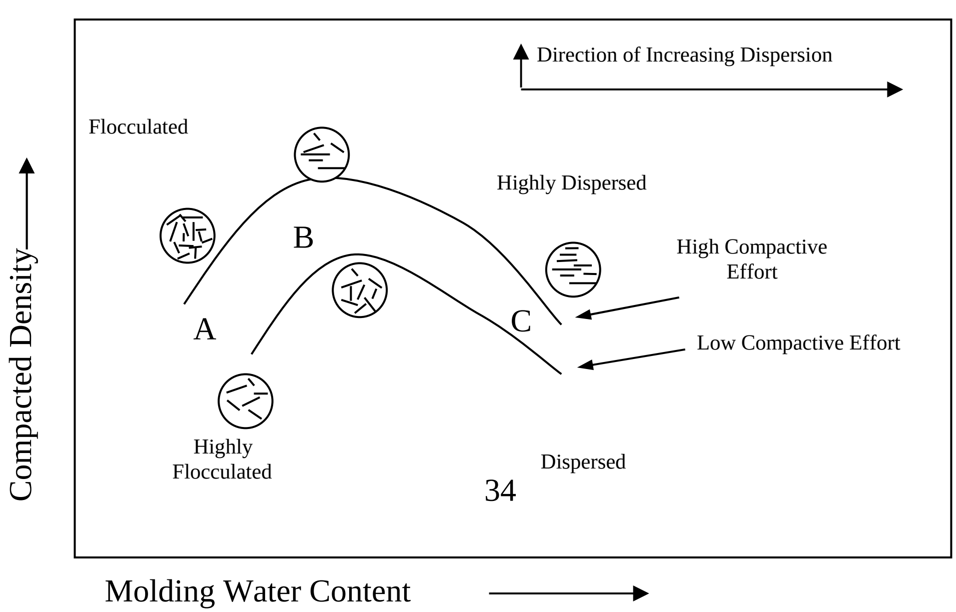


Figure 2.16 Structure of Compacted Clays (from Lambe, 1958)

Engineering Properties of Compacted Clays

Strength

The strength of compacted clays depends upon:

1. Particle spacing: As the particle spacing decreases, the unit weight increases, and consequently the strength also increases.
2. Water content: As the water content, "w," increases, the strength decreases.
3. Particle orientation: A flocculated structure is stronger than a dispersed structure.

For example, comparing the strengths of Points A and C:

1. The particle spacing of both A and C is the same; they both have the same unit weight.
2. The water content of A is drier than C, hence A is stronger than C.
3. In terms of particle orientation; A is flocculated, and consequently stronger than the dispersed orientation of C.

In conclusion, based on the above comparisons, A is deemed stronger than C.

Continuing, let's now compare Points A vs. B.

1. The particle spacing of B is greater than A, hence B is stronger than A
2. The water content of B is greater than A, thus A should be stronger than B.
3. In terms of particle orientation, A is more flocculated than B, thus A should be stronger than B

However, this conclusion that for 2 out of 3 comparisons, A is stronger than B suggests that compaction to optimum conditions is detrimental. Since this conclusion is illogical, a hierarchy must exist among the 3 criteria, that is: spacing > water content > orientation.

An illustration of this hierarchy is presented in Figure 2.17, which shows that for w < 15%, the strength increases with density. However, for w > 15% (wet side of optimum) an increase in density does not equate to an increase in strength. Consequently, higher densities do not equate to higher strength. For the dry side strength increases rapidly with density, but for the wet side, strength decreases with increasing density; i.e., w controls, not γ.

If a compacted soil becomes saturated (soaked), the strength concept changes due to swelling causing a decrease in unit weight. Figure 2.18 (Hausmann, 1990) shows the effect if wetting occurs after compaction: (1) If swelling is allowed, then γ_d decreases and w increases; therefore, the strength decreases due to lower density and higher water content. Figure 2.18 shows that the unsoaked CBR has the highest strength on the dry side of optimum. However, after soaking, the highest strength is achieved at optimum conditions, and the "dry" side has suffered a considerable loss in strength. At optimum, the density is the highest and consequently the permeability is low leading to less effect due to soaking. Note that for the wet side of optimum, the as molded and soaked specimens have equal CBR's, thus soaking has little effect on the wet side of optimum.

Swelling Potential

The effects of density and water content on swelling are also presented in Figure 2.18 and Figure 2.19 (Hausmann, 1990) The percent swell is a measure of free swell, whereby an oedometer specimen is saturated and allowed to swell under a very small surcharge. These figures show that the dry side of optimum is swells the most. The sample wet of optimum exhibit little swelling potential. Considering that the double water layer is satisfied on the wet side of optimum there is little thrust by the clay minerals to imbibe water. Consequently, the swell potential is small. Conversely, dry of optimum, the clay imbibes water thickening the double water layer and causing increased swelling. At optimum conditions, the proximity of the clay particles is also conducive to swell as the double water layer thickening expands the particles apart. However, at optimum, this expansion is tempered by a lower permeability and thicker water layer than dry of optimum.

Thus to minimize expansion, the soil should be compacted wet of optimum at a low density. At this condition caution must be given as a lower strength results.

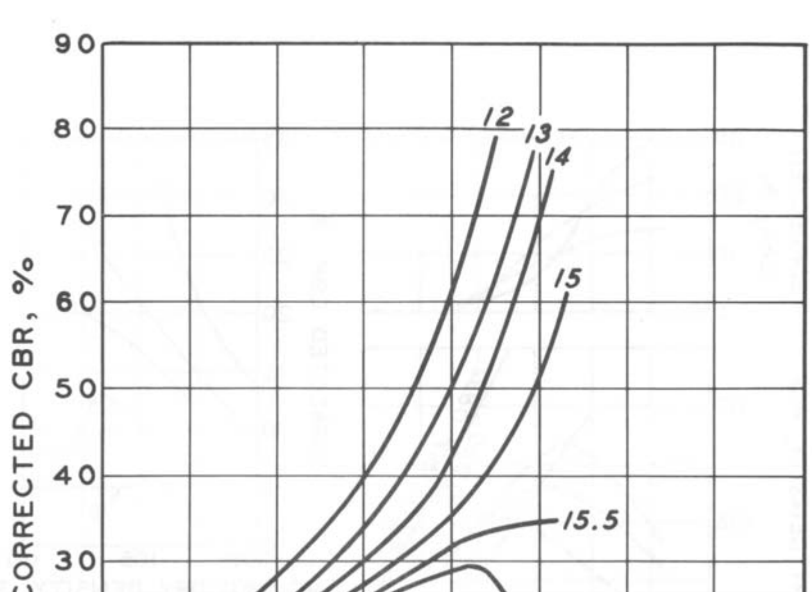


Figure 2.17 Effects of Density and Water Content on CBR Strength

(from USAE-WES, 1976)

Permeability

Figure 2.20 illustrates the effects of density and water content on permeability. As shown, permeability depends upon:

1. Particle spacing. As particle spacing decreases, also permeability decreases; i.e., denser is less permeable.
2. Water content. As water content increases, permeability decreases; i.e., wet of optimum is less permeable.
3. Particle orientation. The “open” flocculated structure is more permeable than dispersed.

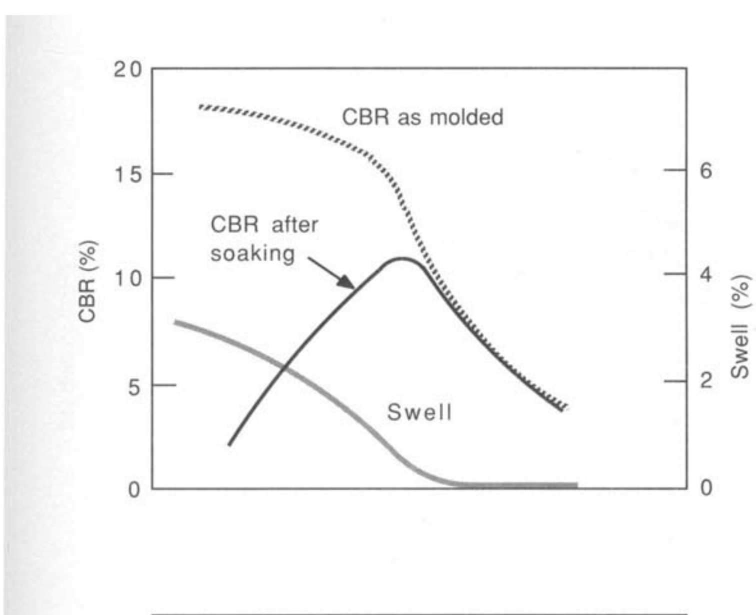


Figure 2.18 Density and CBR for a Silty Clay

(from Hausmann, 1990)

For example, comparing Point A vs. B in Figure 2.16,

1. Particle spacing. B is denser than A and hence exhibits a lower permeability.
2. Water content. B is wetter than A and hence exhibits a lower permeability.
3. Particle orientation. B is more oriented than A and hence exhibits a lower permeability.

Conclusion is that B is less permeable than A.

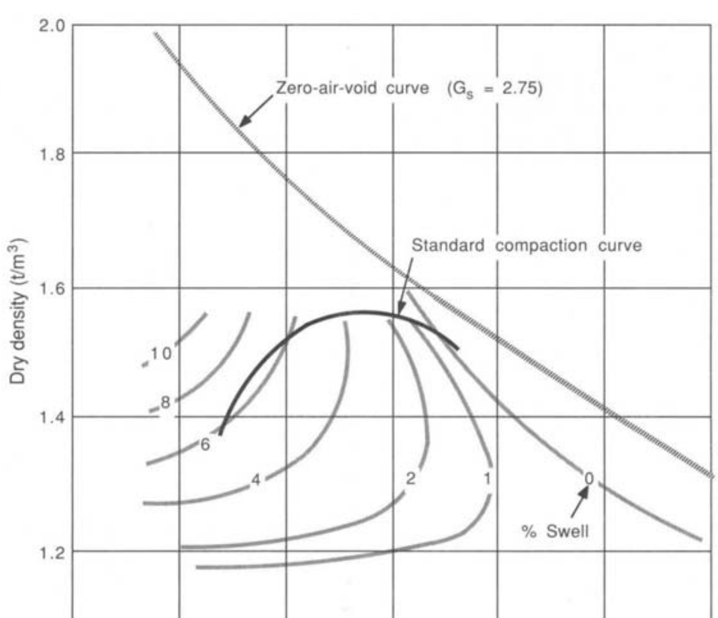


Figure 2.19 Percent Swell vs. Density and Water Content
(from Hausmann,1990 and Holtz and Gibbs, 1956)

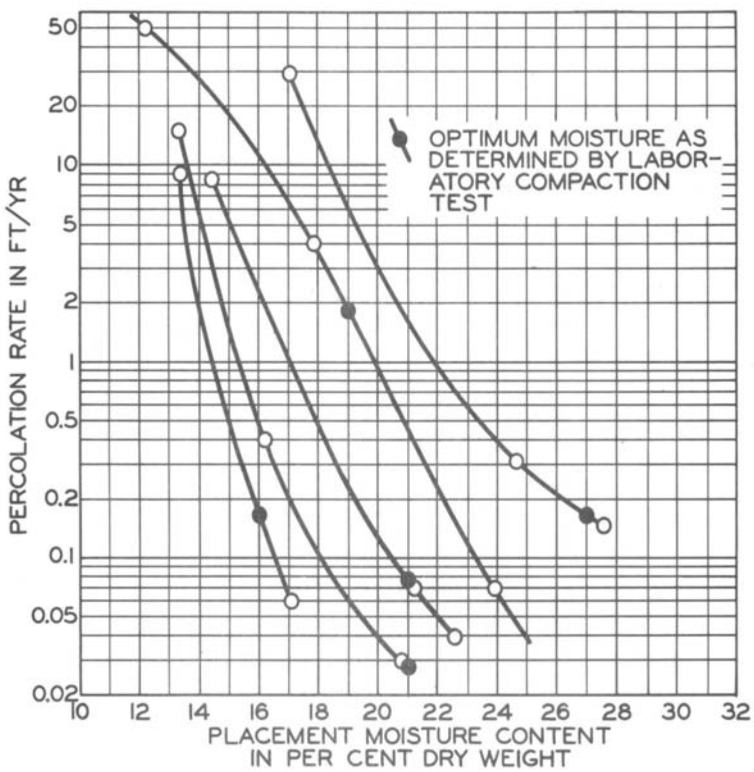


Figure 2.20 Effects of Density and Water Content on Permeability (from USACE, 1976)

For example, comparing Point A vs. C in Figure 2.16

1. Particle spacing. A and C exhibit the same density.
2. Water content. C is wetter than A and thus exhibits a lower permeability.
3. Particle orientation. C is more dispersed than A and hence exhibits a lower permeability.

Conclusion is that C has a lower permeability than A.

In summary, a more oriented (dispersed) has a lower permeability, hence kneading compaction or wet produces lower permeability.

Compressibility

The effects of density and water content on compressibility are shown in Figure 2.21. As shown, the magnitude of the applied load dictates the compression response. If the stresses are high, the *flocculated* structure, dry of optimum, compresses more than the *oriented* structure, wet side of optimum. However, if low stresses are applied, the reverse is observed, and the *dispersed* structure compresses more. Consequently, it depends upon the "bond strength" between flocculated particles. For the “dry” flocculated condition the double water layer is thin, and the forces of attraction are high. If the applied load does not exceed these attractive forces the flocculate structure is able to carry the load. However, once these attractive forces are exceeded, than the flocculated structure collapses and compression occurs.

At optimum conditions, the density is highest; i.e., the void ratio lowest, and compressibility minimized.

On the wet side of optimum, compression of the oriented structure is resisted by the thick incompressible double water layer, and compression is minimal under high loads.

Figure 2.22 (Hausmann, 1990) shows the effects of soaking on compressibility. As discussed previously, dry of optimum has less settlement if left unsoaked. However, when saturated, the flocculated dry side collapses and shows the greatest settlement. In this case, the double water layer imbibes water and thickens causing the attractive forces to diminish and settlement occurs. Conversely, the wet side of optimum exhibits less distress upon saturation as the double water layer is thicker and the permeability wet of optimum is low.

Table 2.4 summarizes the effects of density and water content on the engineering properties of compacted clays.

Method of Compaction

Figure 2.23 (Hausmann, 1990) shows the effect of structure on the stress-strain response. The static compaction lends towards a flocculated structure and has the highest strength and brittle response. Kneading compaction has the greatest shear strain and lends towards a dispersed (oriented) structure, which produces a lower strength and softer response.

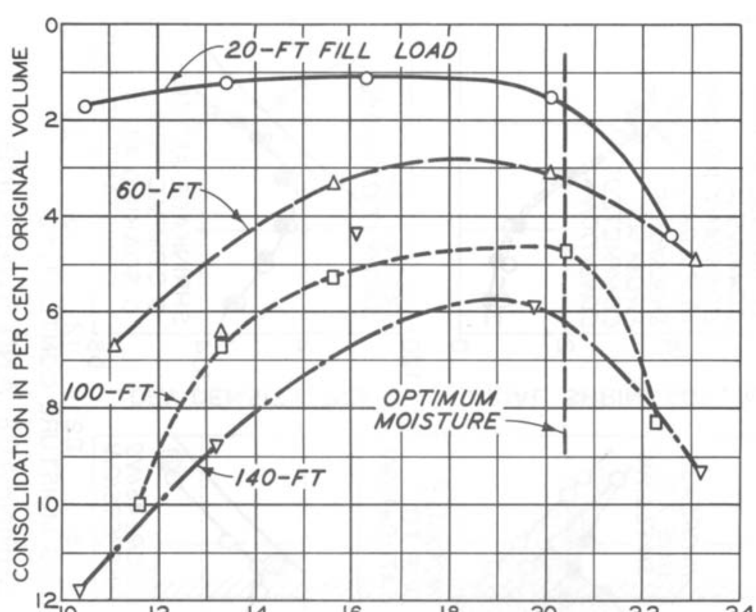


Figure 2.21 A Effect of Applied Stress on Compressibility of Compacted Clay
(from USAE,1976)

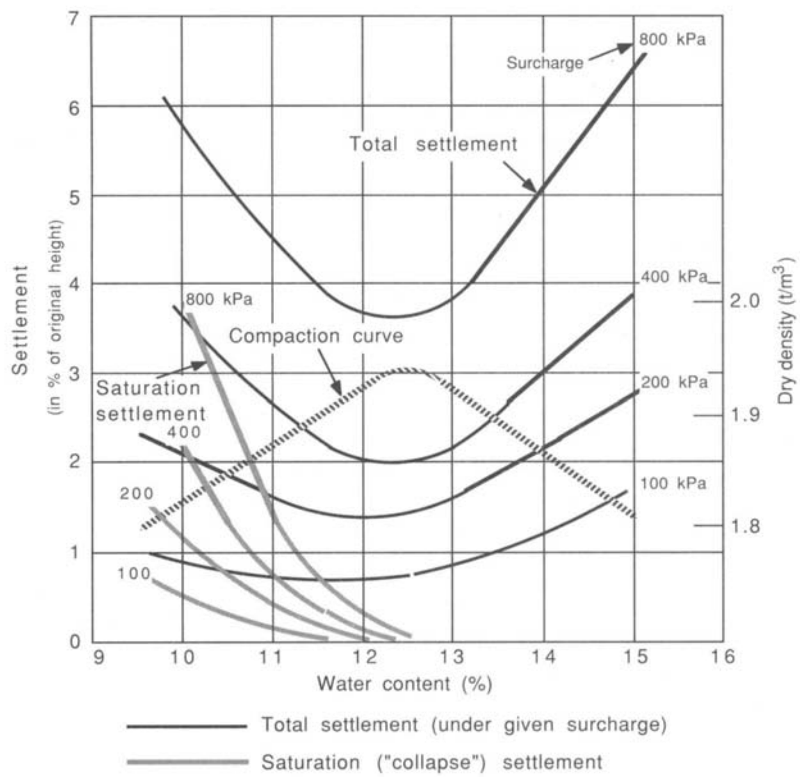


Figure 2.22 Hausmann, 1990, Collapse Settlement and Total Settlement of Compacted Soil
(from Huder, 1964)

Table 2.4 Comparison of Soil Properties Dry or Wet of Optimum
(from Lambe,1958, and Holtz and Kovacs, 1981)

Property	Comparison
1. Structure	
a. Particle Arrangement	Dry – Flocculated (Random)
b. Double Water Layer	Dry side deficient; thus imbibes more water, swells more
c. Permanence	Dry more sensitive to change
2. Strength	

a. a. Permanence	Dry side stronger Dry side swells and loses strength upon saturation
3. Permeability a. Magnitude b. Permanence	 Dry side more permeable Dry side permeability reduced more by permeation
4. Compressibility a. Permanence	 Dry side less compressible under low pressures Dry side collapses when saturated under high loads

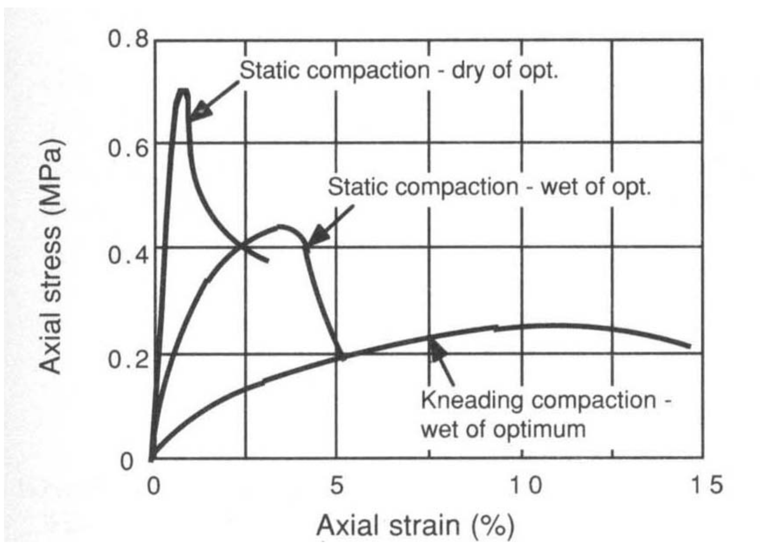


Figure 2.23 Effect of Compaction Method on Unconfined Compressive Strength

(from Hausmann, 1990 and Lee and Haley, 1968)

Figure 2.24 (Seed & Chang, 1959) shows that the method of compaction that produces the greatest shear strain causes a more dispersed structure and lower strength. Static compaction has the most flocculated structure and hence highest strength. However, wet of optimum has lesser effect for the method of compaction.

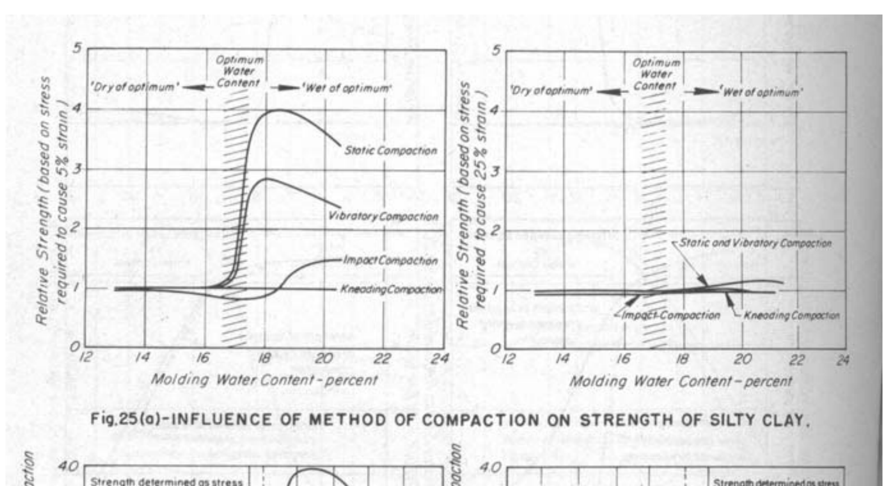


Figure 2.24 Effect of Compaction Method on Strength

(from Seed and Chang, 1959)

Effect of Soil Type

Figures 2.25 and 2.26 illustrate the effects of soil type on compaction curves. As shown in Figure 2.25 the highest density is achieved for the well graded non-plastic soils (loamy non-plastic sand and sandy loam). Conversely, the lowest density is achieved for the poorly graded sand. By explanation, the well graded sands have sufficient fines to fill the inter-grain voids producing greater densities. The lowest density poorly graded sand is a result of insufficient plasticity (LL) to respond to Proctor impact compaction, and increasing water content. For this sand, vibratory compaction is better.

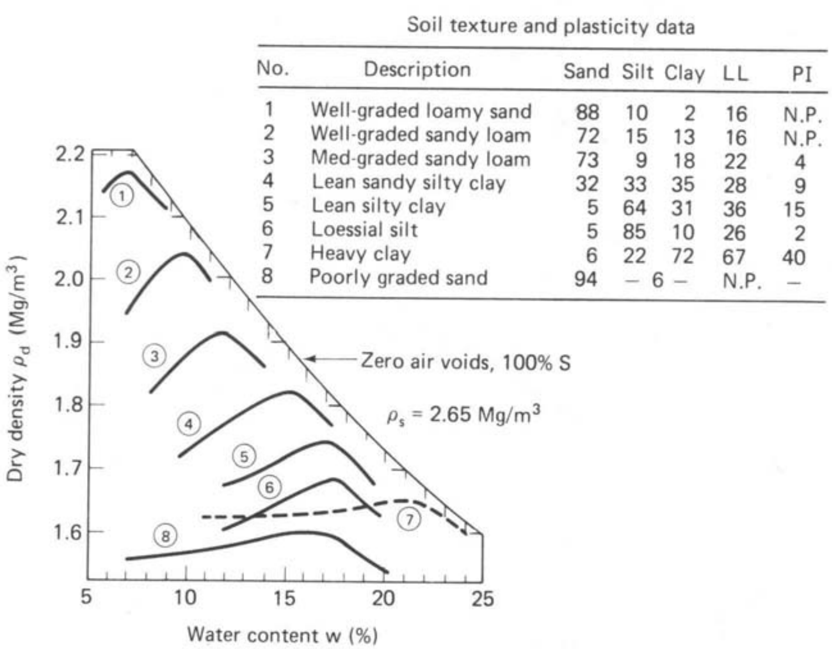


Figure 2.25 Effect of Soil Type on Compaction Curves (after Johnson and Salberg, 1960; from Holtz and Kovacs, 1981)

Figure 2.24 also shows that increasing fines content and plasticity generally results in lower densities and obviously higher optimum water contests. Here we see the double water layer with a specific gravity of 1.00 creating both lower weight and also causing greater voids. Both actions lead to lower densities.

Figure 2.25 illustrates the effect of soil type on the shape of compaction curves. The sharpest peaked curves are for low plasticity clays and silts. For these soils, the double water layers are not thick due to the low negative charge on the clay particle not influencing greatly the dipolar water molecules. Consequently, only a few percentage points change from “dry” to “wet” of optimum. This water sensitivity wreaks havoc with controlling field compaction and can lead to controversy in following specified field conditions. The “fat” clay curve with thicker double water layers illustrates a water insensitivity, but lower density.

Also illustrated in Figure 2.25 is a compaction curve for sand. As shown, non-plastic sands do not develop a double water layer and consequently impact compaction in ineffective. By explanation, the curve shape is a reflection of the capillary tension created by the moisture films between the sand grains. The highest density occurs at zero water content (“bone- dry”), and the gradual addition of water creates capillary tension films, which resist the impact compactive effort causing the sand to “fluff” or bulk. Consequently, field conditions that minimize capillary films produce more efficient compaction; these are completely dry or saturated.

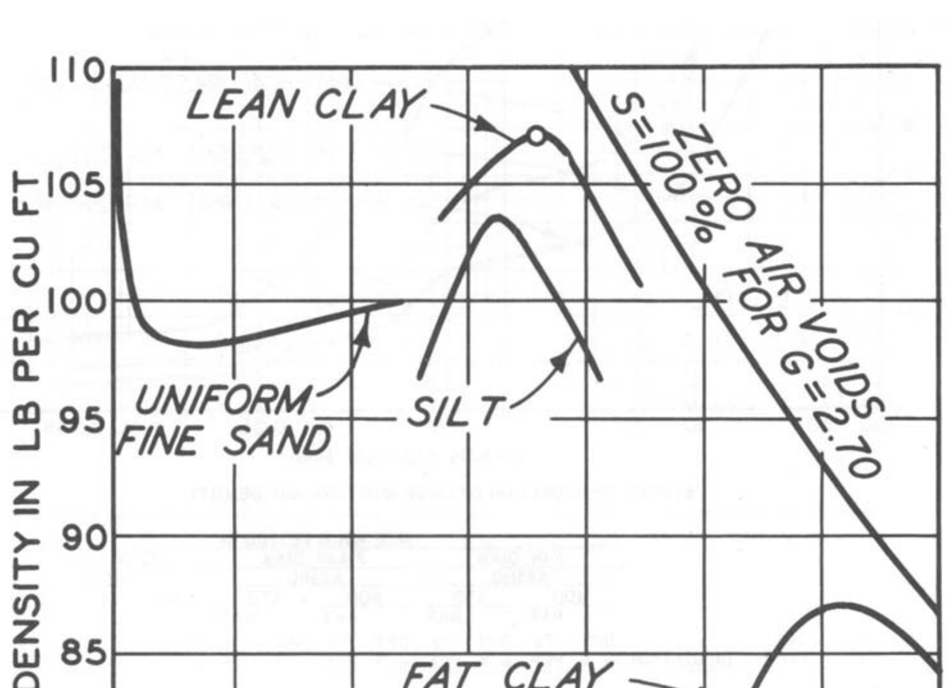


Figure 2.26 Effect of Soil Type on the Shape of Compaction Curves

(from USACE-WES, 1976)

Field Considerations (from USACE-WES, 1976)

WES conducted compaction studies on a lean clay (PI = 13) using a test section consisted of five units, having different water contents bracketing the laboratory optimum water content. Tests were made with a sheepsfoot roller loaded to result in nominal foot pressures of 250, 500, and 750 psi. Test fills were constructed to a height of 30 in. in five layers 6 in. thick after compaction.

Results -

1. Densities obtained from various lifts showed equal densities for 250-, 500-, and 750- psi rollers. Heavier rollers exceeded the bearing pressure of soil and merely penetrated into soil until unit pressure equaled bearing capacity of the soil. The conclusion is that foot pressures should not exceed soil bearing capacity. Visual observation reveals that roller should walk-out after 4-8 passes otherwise bearing capacity exceeded.
2. Lift thickness should not exceed shank length, which is about 7 inches.
3. Figure 21 shows that field compaction curves mirror lab curves, which provides confidence that laboratory and field compaction responses are similar.

The CBR data were developed with a sheepsfoot roller having a nominal foot pressure of 250 psi, equipped with feet having 7-sq-in.contact area, and using 12 passes of the roller. (The data in the top left-hand plot of Figure 2.27) The data shown on the right of Figure 2.27 are similar except that the roller was equipped with 21-sq-in. feet, and the material was subjected to 24 passes of the roller. The strength is highest for the dry side of optimum and decreases with increasing water content. More passes cause a higher density and subsequently higher CBR strength. However, it was found that for the compaction water contents at or dry of the line of optimum water contents, an increase in density resulted in an increase in CBR. However, at water contents above the field optimum for a given compaction effort, an increase in density produced a decrease in CBR.

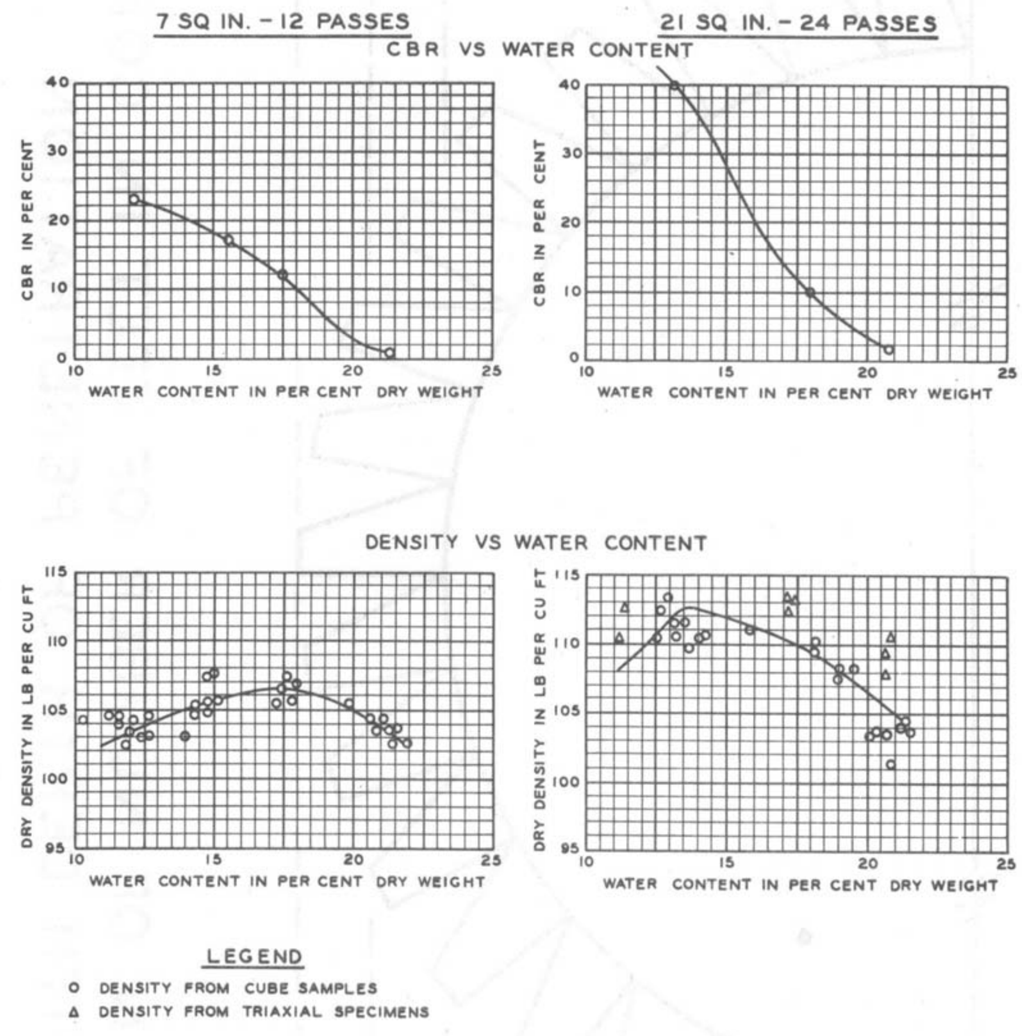


Figure 2.27 Typical Data from Field Compaction Tests

Figure 2.28 shows the effects of foot size and number of passes on density and optimum water content. These plots make use of an additional variable (E), which is considered to be an index of the compaction effort and is the product of foot size and passes divided by 42 (42 sq. in. is the ground area covered by one 7-sq.-in. foot in 6 passes, and results in a value of unity for the lowest effort used). For example energy = 2 is equivalent to either 6 passes of a 14 in^2 foot roller or 12 passes 7 in^2 foot roller. As expected, as the compactive energy (number of passes and foot size) increases, density increases and optimum water content decreases.

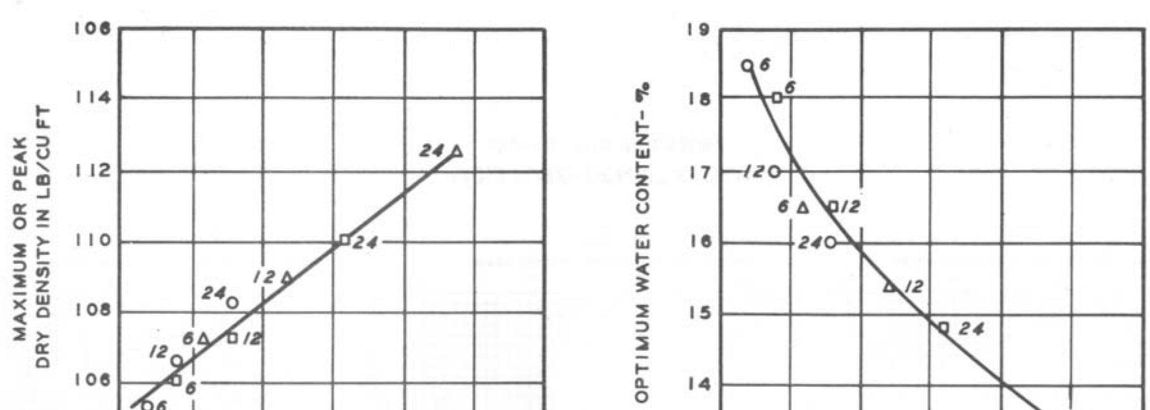


Figure 2.28 Effect of Foot Size and Energy on Compaction

WES evaluated compaction using rubber- tired rollers Test sections were built in 6-in.-thick compacted layers, which were compacted by wheel loads of 10,000, 20,000, and 40,000 lb, respectively, using 4, 8, and 16 coverages of the roller.

Results –

1. Wheel load. Initially, it was thought that an increase in wheel load should result in an increase in density. However, after the fills were sampled and the test data analyzed, it was revealed that essentially the same densities were obtained with the three different wheel loads. In these tests, the tire-inflation pressure was maintained constant at about 65 psi. In analyzing the data, it was determined that the actual contact pressure being exerted on the soil was essentially the same for the three different wheel loads.

   As alluded to in Figure 2.29 the *contact pressure* not *wheel load* controls density for equal lift thicknesses. As noted in this plate, the stress decreases quite rapidly with depth, and the difference in contact pressure at a 6-in, depth between the 10,000- and 40,000-lb

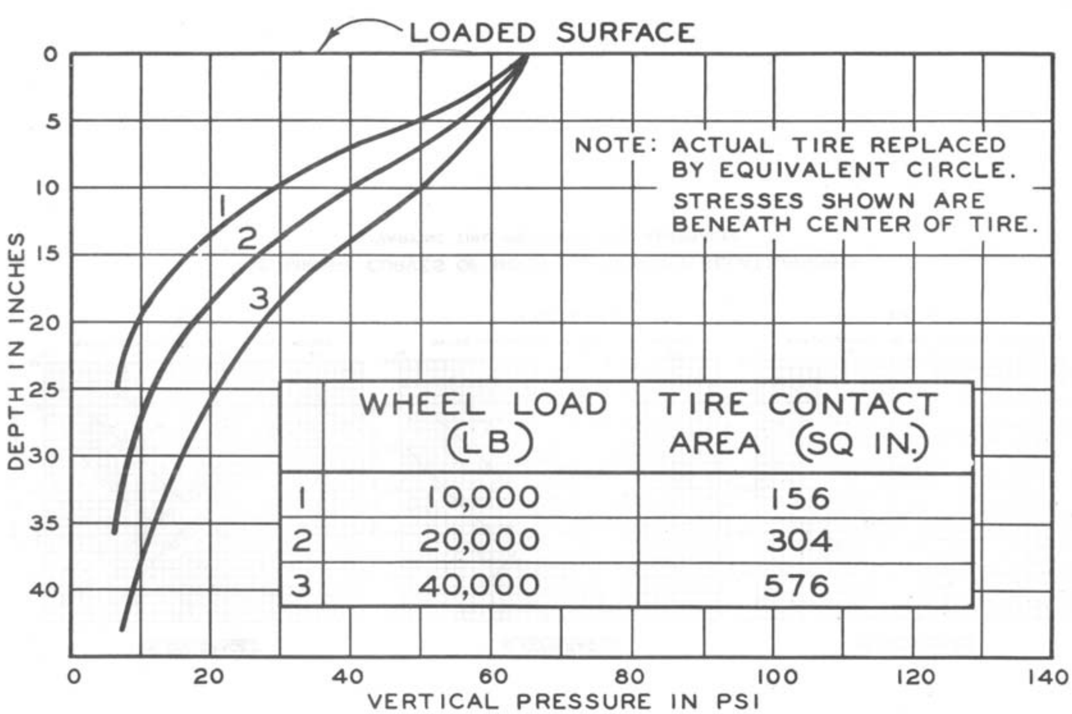


Figure 2.29 Pressure Distribution Beneath Wheel Load with 65-psi Tire Pressure

wheel load is relatively small. Therefore, the average pressure exerted in the 6-in, compacted layer was essentially the same for the three different wheel loads. In other tests with rubber-tired rollers, it has been determined in compacting thick lifts that an increase in wheel load is quite beneficial. For example, the 10,000-lb wheel load would only be exerting a pressure of 10 psi. at a 20-in, depth, whereas a 40,000-lb wheel load would be exerting a pressure of approximately 30 psi or almost three times as much at a 20-in. depth. Therefore, an increase in wheel load is quite effective at greater depths but has very little effect in 6-in.-thick lifts.

2. Figure 2.30 shows effects of tire pressure, water content, and number of passes. For each family of curves, note that a substantial increase in density was obtained with an increase in tire-inflation pressure. Also, by comparing the different plots, it can be noted that a slight increase in density was obtained by increasing the number of coverages from 4, to 8, to 16. The three separate plots represent the density obtained at three different water contents: 13 percent (top plot), 16.3 percent (center plot), and 18 percent (lower plot) molding water contents. These data have a practical application in selecting a tire pressure for economical operation in the field. For example (see center plot), it a specification requires that a soil be compacted at 16.3 percent water content to a density of 110 lb per cu ft, the data indicate that the density can be obtained by 16 coverages of a roller with a tire-inflation pressure of about 80 psi, or by 8 coverages of the same roller with tires inflated to about 90 psi, or by 4 coverages of the same roller with tires inflated to 110 psi. This illustrates the significant effect of the tire-inflation pressure on the density obtained.

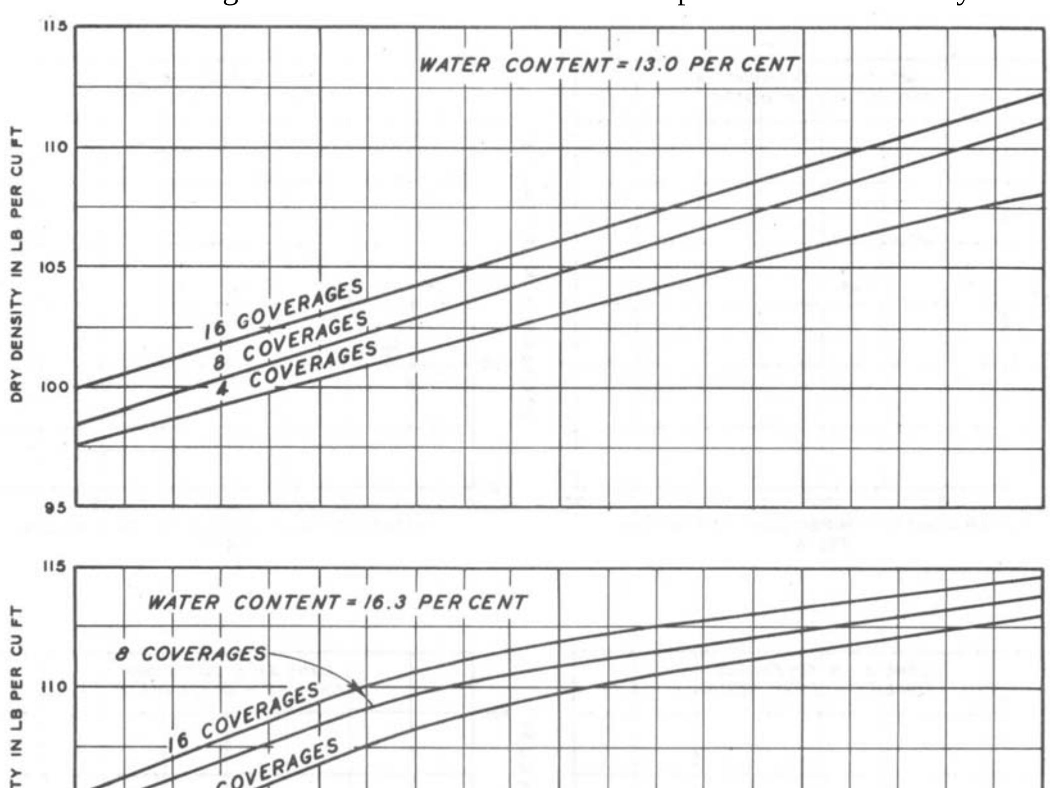


Figure 2.30 Effect of Tire Pressure and Coverages on Density

However, note that compaction dry of optimum (13%) is less efficient than at optimum (16.3). For the same illustration to achieve a density of 110 pcf, 16 coverages requires a tire pressure of 130 psi vs. 80 psi. Also wet of optimum, is the most efficient, as a tire pressure of only 75 is required to achieve a density of 110 pcf. In conclusion, compaction dry of optimum is least efficient as the double water is incomplete and the attractive forces resist "particle sliding" to a denser configuration.

Comparison of Field and Laboratory Compaction

A comparison of optimum water content and maximum density developed in laboratory tests with that developed in field tests with rubber-tired rollers on the lean-clay soil is shown in Figure 2.31. These data show that for equal densities the optimum water content developed in the field was slightly higher than the optimum water content developed in the laboratory, the difference being in the order of 0.5 to slightly over 1.0 percent at the lower densities. Similar comparisons of optimum water content developed by sheepsfoot rollers usually fall slightly to the left of the laboratory optimum. However, the optimums developed in the field tests are quite close to the optimums developed in the laboratory. For control purposes in the field, the laboratory optimums are adequate.

Conclusions - By selection of the proper type of roller for the soil being compacted and a knowledge of the different variables affecting compaction, adequate compaction can be obtained in the field to provide the necessary stability and to prevent detrimental settlement in earth fills and embankments. For cohesive materials, maximum density is obtained at a water content near the laboratory optimum. For cohesionless materials, i.e., materials that are free-draining, maximum density is obtained with the materials in a saturated condition at the start of compaction. The degree of compaction obtained is a function of compaction effort. An increase in compaction effort for the proper moisture condition will result in an increase in the maximum density. An increase in compaction effort can be obtained by an increase in contact pressure and an increase in number of coverages or a decrease in thickness of lift of the material being compacted. A combination of these variables for use on any given job will depend upon the difficulty of compaction, the degree of compaction required, and the economics of obtaining the desired results.

Field Compaction Considerations (US Army Corps of Engineers EM-1911, 1995)

a) Evaluation of test results *and subsequent actions to be taken.* As soon as field test results are obtained, they must be compared to appropriate values of maximum dry density and optimum water content to determine if specification requirements have been met. If measured values match or exceed specification requirements, the next lift can be added. If test results show that specifications have not been met, corrective measures must be taken immediately. A lift must be rejected if the material is too wet or too dry. If density is too low but water content is acceptable, additional rolling is all that is required. If, however, water content is outside specifications, the entire lift should be reworked and rerolled. A lift that is too wet should be worked by disking until the water content is lowered to an acceptable value and then recompacted. A lift that is too dry should be disked, sprinkled, and redisked until the additional water is uniformly distributed, then recompacted. It is important when reworking a rejected lift that the full lift depth be reworked, not just the upper portion. All reworked lifts should be retested for density and water content. It is desirable to determine the reason(s) for an unsatisfactory lift in either borrow or fill operations, so that conditions causing the problem may be corrected on future lifts.

b) Rain water. If a sheepsfoot roller is used for general compaction, smooth-wheel rollers (steel or rubber) can be employed to seal the surface when rain is imminent. In any event, the construction surface should be kept sloped to allow the water to run off instead of standing in puddles and soaking in. After a rain, if some ponding does occur, it is usually easy for the contractor to install a few small ditches to drain these areas. (3) It is often necessary after a rain to scarify and work the construction surface to a depth below that of excessive moisture penetration until it is dried to a satisfactory water

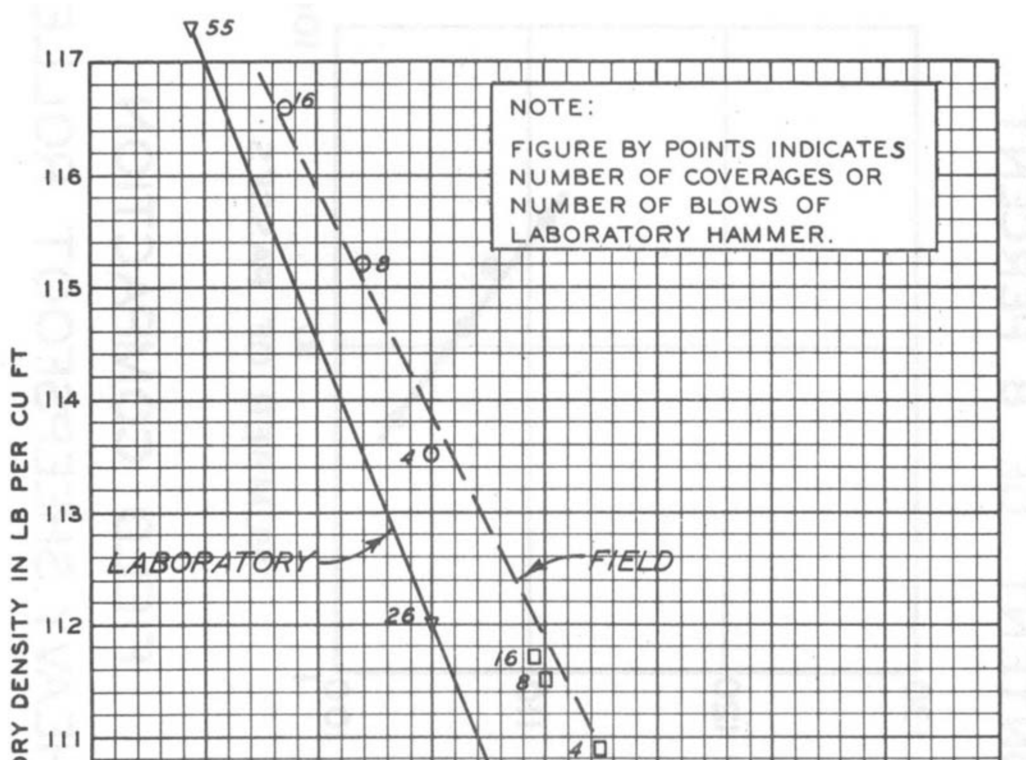


Figure 2.31 Plate 16 Field vs. Laboratory Compaction Conditions

content or, to remove and waste all affected material. If procedures to facilitate runoff are followed (sloping the surface, sealing the surface with smooth rollers, etc.), the depth of moisture penetration will be kept to a minimum.

c. Dry weather. (1) If material being dumped on the fill is too dry for proper compaction, water must be added by sprinkling after it is spread and before it is rolled. The amount of water added and the blending required will depend on grain size and plasticity of the soil, fine-grained soils of high plasticity requiring the greatest amount of blending. Soil must be worked with disks to thoroughly blend and homogenize added water into the soil. The importance of uniform moisture distribution cannot be overemphasized; if pockets of wet and dry soil are allowed in uncompacted material, very poor compaction will result. (2) Sprinkling the soil can be accomplished by hosing from a pipeline, located along either the embankment toe or the crest, or by the use of water trucks. The latter method is the most effective and the most commonly used today.

Pressure sprinkling systems on trucks are superior to gravity systems and should be employed if at all possible. Water sprays must not be directed on the soil with such force as to cause fines to be washed out. Until the inspectors and contractor personnel have gained a "feel" for the amount of water needed, rough computations of the number of gallons to add for a given area should be made, and water applied accordingly. After a few trials, a feel for the proper amount will develop. The coarser and less plastic the soil, the more easily water can be added and worked uniformly into it. It is very difficult to obtain uniform water content distribution in plastic clays containing lumps without a “curing” period of a few days; this is, of course, not practical on the embankment surface. Consequently, disking followed by addition of water and then thorough mixing with a heavy rotary pulverizer may be required to obtain uniform distribution of water in such soils.

SPECIFICATIONS

It is important to recall that the objectives of compaction are to improve the: (1) shear strength, (2) decrease permeability, (3) increase stiffness and reduce settlement, etc. However, these engineering properties are not measured in the field due to expense and difficulty. Consequently, it is assumed that by specifying field density and water content requirements will achieve these desired engineering characteristics.

There are basically two types of compaction specifications: (1) end-result, and (2) method.

An end result specification usually specifies a certain percent compaction; i.e., 95 % standard Proctor. A more detailed specification will include a range of water contents; i.e., ± 2% of optimum, and/or a minimum lift thickness.

A method specification may specify the: weight and type of roller, lift thickness, placement water content, and minimum number of passes, or various combinations of these. For this specification, the responsibility rest upon the owner, and often test fills are made to establish these specification details.

Example – Corps of Engineers Guide Specification

The US. Army Corps of Engineers guide specifications are listed at: http://www.hnd.usace.army.mil/techinfo/gspec.htm. SECTION 02300a Page 71.4.4 Degree of Compaction specifically addresses compaction issues; from which, several excerpts follow.

> Degree of compaction required, except as noted in the second sentence, is expressed as a percentage of the maximum density obtained by the test procedure presented in ASTM D 1557abbreviated as a percent of laboratory maximum density. Since ASTM D 1557 applies only to soils that have 30 percent or less by weight of their particles retained on the 9.0 mm 3/4 inch sieve, the degree of compaction for material having more than 30 percent by weight of their particles retained on the 9.0 mm 3/4 inch sieve shall be expressed as a percentage of the maximum density in accordance with AASHTO T 180 Method D and corrected with AASHTO T 224. To maintain the same percentage of coarse material, the "remove and replace" procedure as described in the NOTE 8 in Paragraph 7.2 of AASHTO T 180 shall be used.
>
> 3.6 BACKFILL

Backfill adjacent to any and all types of structures shall be placed and compacted to at least 90 percent laboratory maximum density for cohesive materials or 95 percent laboratory maximum density for cohesionless materials to prevent wedging action or eccentric loading upon or against the structure. Ground surface on which backfill is to be placed shall be prepared as specified in paragraph PREPARATION OF GROUND SURFACE FOR EMBANKMENTS. Compaction requirements for backfill materials shall also conform to the applicable portions of paragraphs PREPARATION OF GROUND SURFACE FOR EMBANKMENTS, EMBANKMENTS, and SUBGRADE PREPARATION, and Section 02630 STORM-DRAINAGE SYSTEM; and Section 02316 EXCAVATION, TRENCHING, AND BACKFILLING FOR UTILITIES SYSTEMS. Compaction shall be accomplished by sheepsfoot rollers, pneumatic-tired rollers, steel-wheeled rollers, vibratory compactors, or other approved equipment.

3.7 PREPARATION OF GROUND SURFACE FOR EMBANKMENTS

3.7.1 General Requirements

Ground surface on which fill is to be placed shall be stripped of live, dead, or decayed vegetation, rubbish, debris, and other unsatisfactory material; plowed, disked, or otherwise broken up to a depth of [_____]; pulverized; moistened or aerated as necessary; thoroughly mixed; and compacted to at least 90 percent laboratory maximum density for cohesive materials or 95 percent laboratory maximum density for cohesionless materials. Compaction shall be accomplished by sheepsfoot rollers, pneumatic-tired rollers, steel-wheeled rollers, vibratory compactors, or other approved equipment. The prepared ground surface shall be scarified and moistened or aerated as required just prior to placement of embankment materials to assure adequate bond between embankment material and the prepared ground surface.

3.8 EMBANKMENTS

3.8.1 Earth Embankments

****

NOTE: Moisture content limits for compaction should be included in these paragraphs when necessary for obtaining strength and stability in embankments and fill, for controlling movement of expansive soils and when, in the opinion of the project geotechnical engineer, moisture control is required for the soils being used.

****

Earth embankments shall be constructed from satisfactory materials free of organic or frozen material and rocks with any dimension greater than 75 mm 3 inches. The material shall be placed in successive horizontal layers of loose material not more than [_____] millimeters inches in depth. Each layer shall be spread uniformly on a soil surface that has been moistened or aerated as necessary, and scarified or otherwise broken up so that the fill will bond with the surface on which it is placed. After spreading, each layer shall be plowed, disked, or otherwise broken up; moistened or aerated as necessary; thoroughly mixed; and compacted to at least 90 percent laboratory maximum density for cohesive materials or 95 percent laboratory maximum density for cohesionless materials. Compaction requirements for the upper portion of earth embankments forming subgrade for pavements shall be identical with those requirements specified in paragraph SUBGRADE PREPARATION. Compaction shall be accomplished by sheepsfoot rollers, pneumatic-tired rollers, steel-wheeled rollers, vibratory compactors, or other approved equipment.

Testing shall be performed by an approved commercial testing laboratory or by the Contractor subject to approval. If the Contractor elects to establish testing facilities, no work requiring testing will be permitted until the Contractor's facilities have been inspected and approved by the Contracting Officer. Field in-place density shall be determined in accordance with [ASTM D 1556] [ASTM D 2167] [ASTM D 2922]. [When ASTM D 2922 is used, the calibration curves shall be checked and adjusted using only the sand cone method as described in ASTM D 1556. ASTM D 2922results in a wet unit weight of soil and when using this method ASTM D 3017 shall be used to determine the moisture content of the soil. The calibration curves furnished with the moisture gauges shall also be checked along with density calibration checks as described in ASTM D 3017; the calibration checks of both the density and moisture gauges shall be made at the beginning of a job on each different type of material encountered and at intervals as directed by the Contracting Officer.] [ASTM D 2937, Drive Cylinder Method shall be used only for soft, fine-grained, cohesive soils.] When test results indicate, as determined by the Contracting Officer, that compaction is not as specified, the material shall be removed, replaced and recompacted to meet specification requirements. Tests on recompacted areas shall be performed to determine conformance with specification requirements. Inspections and test results shall be certified by a registered professional civil engineer. These certifications shall state that the tests and observations were performed by or under the direct supervision of the engineer and that the results are representative of the materials or conditions being certified by the tests. The following number of tests, if performed at the appropriate time, will be the minimum acceptable for each type operation.

3.13.2 In-Place Densities

One test per [_____] square meters, feet, or fraction thereof, of each lift of fill or backfill areas compacted by other than hand-operated machines.

One test per [_____] square meters, feet, or fraction thereof, of each lift of fill or backfill areas compacted by hand-operated machines.

One test per [_____] linear meters, feet, or fraction thereof, of each lift of embankment or backfill for [roads] [airfields].

One test per [_____] linear meters, feet, or fraction thereof, of each lift of embankment or backfill for railroads.

3.13.3 Check Tests on In-Place Densities

SECTION 02300a Page 20

If ASTM D 2922 is used, in-place densities shall be checked by ASTM D 1556 as follows:

One check test per lift for each [_____] square meters, feet, or fraction thereof, of each lift of fill or backfill compacted by other than hand-operated machines.

One check test per lift for each [_____] square meters, feet, of fill or backfill areas compacted by hand-operated machines.

One check test per lift for each [_____] linear meters, feet, or fraction thereof, of embankment or backfill for [roads] [airfields].

One check test per lift for each [_____] linear meters, feet, or fraction thereof, of embankment or backfill for railroads.

3.13.4 Moisture Contents

In the stockpile, excavation, or borrow areas, a minimum of two tests per day per type of material or source of material being placed during stable weather conditions shall be performed. During unstable weather, tests shall be made as dictated by local conditions and approved by the Contracting Officer.

3.13.5 Optimum Moisture and Laboratory Maximum Density

Tests shall be made for each type material or source of material including borrow material to determine the optimum moisture and laboratory maximum density values. One representative test per [_____] cubic meters yards of fill and backfill, or when any change in material occurs which may affect the optimum moisture content or laboratory maximum density.

FDOT 120 Compaction Specifications

http://www11.myflorida.com/specificationsoffice/JULY2002WB.htm#SUPPLEMENTAL SPECIFICATIONS

120-8.2 Dry Fill Method:

120-8.2.1 General: Except as provided below for material placed on unstable ground and for materials used for flattening slopes, construct embankments in successive layers of not more than 6 inches [150 mm] compacted thickness, for the full width of the embankment. Alternately, construct embankments using thick lift construction in successive layers of not more than 12 inches [300 mm] compacted thickness, having demonstrated with a successful test section, the possession and control of compacting equipment sufficient to achieve density required by 120-10.2 for the full depth of a thicker lift, and if the Engineer approves the compaction effort. Notify the Engineer prior to beginning construction of a test section. Construct a test section of the length of one LOT. Perform five QC tests at random locations within the test section. All five tests must meet the density required by 120-10.2 and be verified by the Engineer. Identify the test section with the compaction effort and soil classification in the Density Log Book. In case of a change in compaction effort or soil classification, failing QC test or when the QC tests cannot be verified, construct a new test section. The Contractor may elect to place material in 6 inches [150 mm] compacted thickness at any time. Construct all layers approximately parallel to the centerline profile of the road. The Engineer reserves the right to terminate the Contractor's use of thick lift construction. Whenever the Engineer determines that the Contractor is not achieving satisfactory results, revert to the 6 inch [150 mm] compacted lifts. As far as practicable, distribute traffic over the work during the construction of embankments so as to cover the maximum area of the surface of each layer. Construct embankment in the dry whenever normal dewatering equipment and methods can accomplish the needed dewatering.

120-9 Compaction Requirements.

120-9.1 Moisture Content: Compact the materials at a moisture content such that the specified density can be attained. If necessary to attain the specified density, add water to the material, or lower the moisture content by manipulating the material or allowing it to dry, as is appropriate.

120-9.2 Compaction of Embankments:

120-9.2.1 General: Uniformly compact each layer, using equipment that will achieve the required density, and as compaction operations progress, shape and manipulate each layer as necessary to ensure uniform density throughout the embankment.

120-9.2.3 Compaction Where Plastic Material Has Been Removed: Where unsuitable material is removed and the remaining surface is of the A-4, A-5, A-6, or A-7 Soil Groups (see Florida Sampling and Testing Methods, M145), as determined by the Engineer, compact the surface of the excavated area by rolling with a sheepsfoot roller exerting a compression of at least 250 psi [1.7 MPa] on the tamper feet, for the full width of the roadbed (subgrade and shoulders). Perform rolling before beginning any backfill, and continue until the roller feet do not penetrate the surface more than 1 inch [25 mm]. Do not perform such rolling where the remaining surface is below the normal water table and covered with water. Vary the procedure and equipment required for this operation at the discretion of the Engineer.

120-10 Acceptance Program.

120-10.1 General Requirements:

120-10.1.1 Initial Equipment Comparison: Before initial production, perform a comparison test using the Quality Control, Verifications and Independent Assurance gauges. Unless the Engineer instructs, do not perform the initial equipment comparison more than once per project. When comparing the computed dry density of one nuclear gauge to a second gauge, ensure that the difference between the two computed dry densities does not exceed 2 PCF [32 kg/m3] between gauges from the same manufacturer, and 3 PCF [48 kg/m3] between gauges from different manufacturers. Repair or replace any Quality Control gauge that does not compare favorably with the IA gauge. Perform a comparison analysis between the Quality Control nuclear gauge and the Verification nuclear gauge any time a nuclear gauge or repaired nuclear gauge is first brought to the project. Repair and replace any Quality Control gauge that does not compare favorable with the Verification gauge at any time during the remainder of the project. Calibrate all Quality Control gauges annually.

120-10.1.2 Initial Production Lot: Before construction of any other Lot, prepare an initial control section consisting of one full LOT in accordance with the approved Quality Control Plan for the project. Notify the Engineer at least 24 hours prior to production of the initial control section. When the initial Quality Control test results pass specifications, the Engineer will perform a Verification test to verify compliance with the specifications. Do not begin constructing another LOT until successfully completing the initial production LOT. The Engineer will notify the Contractor of the initial production lot approval within three working days after receiving the Contractor's Quality Control data when test results meet the following conditions: Quality Control tests must meet the specifications. Verification test must meet the specifications. Difference between Quality Control and Verification computed Dry Density results shall meet the requirements of 120-10.1.1. If Verification test result fails the density requirements of 120-10.2, correct the areas of non-compliance. The Quality Control and Verification tests will then be repeated. The Engineer will reject the Contractor's Quality Control Plan after three unsuccessful Verification attempts. Submit a revised Quality Control Plan to the Engineer for approval.

120-10.1.3 Density over 105%: When a computed dry density results in a value greater than 105% of the applicable proctor maximum dry density, the Engineer will perform an Independent Verification density test within 5 feet [1.5 meters]. If the Independent Verification density results in a value greater than 105%, the Engineer will investigate the compaction methods, examine the applicable Maximum Density and material description. The Engineer may collect and test an Independent Verification Maximum Density sample for acceptance in accordance with the criteria of 120-10.2.

120-10.1.4 Quality Control Tests:

120-10.1.4.1 Maximum Density Determination: Determine the Quality Control maximum density and optimum moisture content by sampling and testing the material in accordance with the specified test method listed in 120-10.2.

120-10.1.4.2 Density Testing Requirements: Ensure compliance to the requirements of 120-10.2 by Nuclear Density testing in accordance with FM 1-T 238. Determine the inplace moisture content for each density test. Use Florida

Method FM 1-T 238, FM 5-507 (Determination of Moisture Content by Means of a Calcium Carbide Gas Pressure Moisture Tester), or FM 5-535 (Laboratory Determination of Moisture Content of Granular Soils By Use of a Microwave Oven) for moisture determination. Perform these tests at a minimum frequency of one test per LOT. Determine test locations including Stations and Offsets, using the Random Number generator provided by the Engineer. Do not use note pads or work sheets to record data for later transfer to the Density Log Book. Notify the Engineer upon successful completion of Quality Control testing on each LOT.

120-10.2 Acceptance Criteria:

Obtain a minimum Quality Control (QC) density of 100% of the maximum density as determined by AASHTO T 99, Method C, with the following exceptions: 1) embankment constructed by the hydraulic method as specified in 120-8.3; 2) material placed outside the standard minimum slope as specified in 120-8.2.4; and 3) other areas specifically excluded herein.

120-10.3 Additional Requirements:

120-10.3.1 Frequency: Conduct QC sampling and testing at a minimum frequency listed in the table below. The Engineer will perform Verification sampling and tests at a minimum frequency listed in the table below. Test Name Quality Control Verification Maximum Density One per soil type One per soil type Density One per Lot One per four Lots and the first lift not affected by water.

Test Name	Quality Control	Verification
Maximum Density	One per soil type	One per soil type
Density	One per Lot	One per four Lots and the first lift not affected by water

120-10.4 Verification Comparison Criteria and Resolution Procedures:

120-10.4.1 Maximum Density Determination: The Engineer will verify the Quality Control results if the results compare within 4.5 PCF [72 kg/m3] of the Verification test result. Otherwise, the Engineer will take one additional sample of material from the soil type in question. The State Materials Office or an AASHTO accredited laboratory designated by the State Materials Office will perform Resolution testing. The material will be sampled and tested in accordance with AASHTO T 99, Method C. The Engineer will compare the Resolution Test results with the Quality Control test results. If all Resolution Test results are within 4.5 PCF [72 kg/m3] of the corresponding Quality Control test results, the Engineer will use the Quality Control test results for material acceptance purposes for each LOT with that soil type. If the Resolution Test result is not within 4.5 PCF [72 kg/m3] of the Contractor's Quality Control test, the Verification Test result will be used for material acceptance purposes.

120-10.4.2 Density Testing: When a Verification or Independent Verification density test fails the Acceptance Criteria, retest the site within a 5 feet (1.5 meter) radius and the following actions will be taken: 1. If the Quality Control retest meets the Acceptance Criteria and compares favorably with the Verification or Independent

Verification test, the Engineer will accept those LOTs. 2. If the Quality Control retest does not meet the Acceptance Criteria and compares favorably with the Verification or Independent Verification test, rework and retest the LOT. The Engineer will re-verify those LOTs. 3. If the Quality Control retest and the Verification or Independent Verification test do not compare favorably, complete a new comparison analysis as defined in 120-10.1.2. Once acceptable comparison is achieved, retest the LOTs. The Engineer will perform new verification testing. Acceptance testing will not begin on a new LOT until the Contractor has a gauge that meets the comparison requirements.

120-13 Method of Measurement.

120-13.1 General:

When payment for excavation is on a volumetric basis, the quantity to be paid for will be the volume, in cubic yards [cubic meters], calculated by the method of average end areas, unless the Engineer determines that another method of calculation will provide a more accurate result. The material will be measured in its original position by field survey or by photogrammetric means as designated by the Engineer, unless otherwise specified under the provisions for individual items. Where Subsoil Excavation extends outside the lines shown in the plans or authorized by the Engineer including allowable tolerances, and the space is backfilled with material obtained in additional authorized roadway or borrow excavation, the net fill, plus shrinkage allowance, will be deducted from the quantity of Roadway Excavation or Borrow Excavation to be paid for, as applicable.

REFERENCES

Forssblad, L. (1977) “Vibratory Compaction in the Construction of Roads, Airfields, Dams, and Other Projects,” Research Report No. 8222, Dynapac, S-171, No. 22, Solna, Sweden as referenced in Hausmann (1990).

Gilbert, P. A. (1991) “Rapid Water Content by Computer-Controlled Microwave Drying,” *Journal of Geotechnical Engineering*, Vol. 117, No. 1, pp. 118–138.

Hausmann, M. R. (1990) Engineering Principles of Ground Modification, McGraw-Hill Publishing Company.

Holtz, R.D. and Kovacs, W.D. (1981) An Introduction to Geotechnical Engineering, Prentice Hall, Inc. Englewood Cliffs, NJ.

Holtz, W.G., and Gibbs, H.J. (1956) “Engineering Characteristics of Expansive Clays,” Trans. ASCE.

Huder, J. (1964) “Die Zusammendruckbarkeit des Bodens und deren Bestimmung” Schweizerische Bauzetung, Heft 41, 8 Oct. As referenced in Hausmann, 1990).

Johnson, A.W. and Salberg, J.R. (1960) “Factors that Influence Field Compaction of Soils,” Bulletin 272, Highway research board.

Lambe, T.W. (1958a) "The Structure of Compacted Clay," Journal of the Soil Mechanics and Foundations Division, ASCE, Vol. 84, SM2 pp 1654-1 to 1654-34.

Lambe, T.W. (1958b) "The Engineering Behavior of Compacted Clay," Journal of the Soil Mechanics and Foundations Division, ASCE, Vol. 84, SM2 pp 1655-1 to 1655-35.

Lee, K.L. and Haley, S.C. (1968) "Strength of Compacted Clay at High Pressure," J. Soil Mech. and Fdn. Engr. ASCE Vol. 94 No. SM6, Nov. p 1303-1332.

Meehan and Israngkura (1967) *Canadian Geotechnical Journal*, Vol. IV, No. 3.

Proctor, R.R. (1933) "Fundamental Principles of Soil Compaction," Engineering-News-Record, Vol. 111, Nos. 9, 10, 12, and 13.

Seed, H.B. and Chang, C.K. (1959) "Structure and Strength Characteristics of Compacted Clays," Journal of the Soil Mechanics and Foundations Division, ASCE, Vol. 85, SM 5 pp 87-128.

Troxler (1990) "Surface Moisture Density Gage –3430, Manual of Operation and Instruction" www.troxlerlabs.com.

US Army COE (1995) "Construction and Control for Earth and Rockfill Dams," Engineering Manual 1110-2-1911.

US Army COE (1970) "Laboratory Soils Testing," Engineering Manual (EM) 1110-2-1906.

US Army Engineer Waterways Experiment Station COE (1976) "Notes for Earthwork Construction Inspectors Course," Vicksburg, MS.

Chapter 3

DEEP DYNAMIC COMPACTION

INTRODUCTION

Thus far, compaction as a method of ground modification has been discussed in detail. Traditional compaction has a relatively shallow depth of influence. As the name implies, Deep Dynamic Compaction, DDC, compacts the soil at depth by means of a dynamic impact. In plain terms, a large weight, 5 to 30 tons, is dropped from a height on the order of 40 to 100 feet. This drop imparts a large burst of energy into the ground (force through a distance) as illustrated in Figure 3.1. The drops are performed in a specific pattern and frequency whereby the soil is improved to the desired level.

Figure 3.1 Deep Dynamic Compaction

APPLICATIONS

Dynamic compaction has been successfully used to improve many types of weak ground deposits including:

- Loose naturally occurring soils such as alluvial, flood plain, or hydraulic fill deposits. Landfill deposits both recent and old. Building rubble and construction debris deposits.

- Strip mine spoil. Partially saturated clay fill deposits that are elevated above the water table.
- Collapsible soils including loess.
- Formations where large voids are present such as karst topography or sinkholes that are located close to grade.
- Loose sands and silts to reduce liquefaction potential.
- Special wastes.

An estimated 500 dynamic compaction projects have been completed in the U.S. Most were for commercial purposes. The actual number may be much greater because many projects are not reported in the literature.

A list of highway-related dynamic compaction projects completed in the U.S. was compiled in 1992.(FHWA 1995) Twenty-five projects were identified where dynamic compaction was used on at least a part of the project site. Figure 3.2 indicates where these projects are located and the type of deposit that was densified.

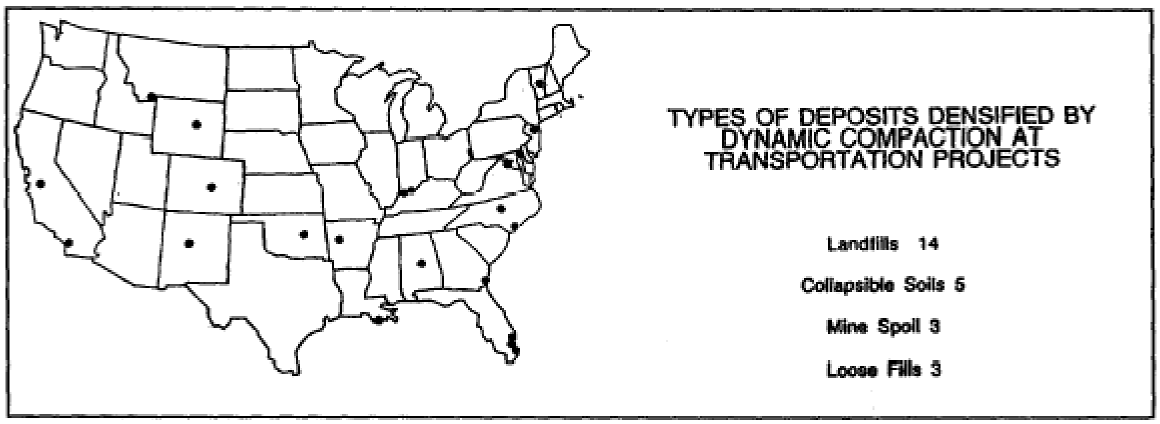


Figure 3.2 Deep Dynamic Compaction in the United States (Drumheller, 1992)

The greatest use of dynamic compaction has been to stabilize former landfills. This is attributed to the need for routing highways through or adjacent to urban sites where the land is at a premium and frequently the only spaces available are sites such as former landfills that have been bypassed for commercial development.

Dynamic compaction has been frequently used to densify collapsible soils present in the western part of the United States. The purpose of densification is to reduce settlement of the pavements that occurs as the soils become wetted tier the highways are constructed.

Mine spoil deposits consisting of reworked shales and sandstones plus soil overburden have also been densified by dynamic compaction. The soil and rock mixture is usually in a medium-dense condition, but often there are pockets of very loose deposits within an otherwise more stable formation. Dynamic compaction has been found to be effective in making the subgrade more uniform.

SUITABILITY

In order to asses the suitability of a site for DDC, several steps are recommended. (FHWA 1995)

1. Categorize soil type. The properties, thicknesses, and extent of the weak ground must be known. Based upon the types of soils that are in need of improvement at the site, the deposits can be rated as favorable, unfavorable, or intermediate for dynamic compaction. Table 3.1 and Figure 3.3 show zones of suitable soil types for DDC.
2. Assess site restraints. The project site should be examined to determine if the ground vibrations or lateral ground displacement could have an effect on adjacent properties. This would be especially important in urban areas where roadways or buildings might be situated in very close proximity to the area to be densified.
3. Determine design requirements. If reduction in settlement is desired, a settlement estimate should be made before and after dynamic compaction and then compared with the requirements of the new embankment or facility. If the settlement is still larger than the new facility can tolerate, an alternate form of site improvement or support should be considered.
4. Estimate costs. A preliminary estimate of costs for dynamic compaction should be made. The cost estimate can be refined later, but a quick cost estimate is necessary to compare with alternate site improvement techniques.

Most Favorable Soil Deposits - Zone 1

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 include pervious granular soils. If these deposits are situated above the water table, densification is immediate as the soil particles are forced into a denser state of packing. If these deposits are situated below the water table, the permeability is sufficiently high, excess pore water pressures generated by the impact of the tamper dissipate almost immediately, and densification is nearly immediate. Pervious granular deposits include not only natural sands and gravels but also fill deposits consisting of building rubble, some mine spoil, some industrial waste fill such as slag, and decomposed refuse deposits as in Figure 3.4.

Dynamic compaction extends the range of compactable soils beyond that which is ordinarily undertaken by conventional compaction. Ordinary roller compaction would be very difficult on some of the coarser grained pervious deposits such as boulders and cobbles, building rubble, or slag deposits.

Unfavorable Soil Deposits - Zone 3

Deposits in which dynamic compaction is not appropriate would clayey soils, either natural or fill, that are saturated. In saturated deposits, improvements cannot occur unless the

Table 3.1 Soil Types for Deep Dynamic Compaction

Steps	Favorable for Dynamic Compaction	Favorable with Restrictions*	Unfavorable for Dynamic Compaction
1. Categorize Soil Type			
Zone 1: Pervious	Best deposit for dynamic compaction	——	——
Zone 2: Semipervious	——	Apply energy in phases to allow for dissipation of pore pressure	——
Zone 3: Impervious	——	Partially saturated impervious soils with deep water	Saturated or nearly saturated impervious soils
2. Assess Site Restraints			
Vibrations	Adjacent to: modern construction, < 19 mm per	19 to 51 mm per sec allowable if adjacent to buildings	Adjacent to: modern construction, > 19 mm per second
Lateral Ground Displacements	Dynamic compaction > 7.6 m from buried utilities	Most buried utilities can tolerate 76 to 127 mm per second	Immediately adjacent to easily damaged
Water Table	> 2 m below grade	< 2 m below grade, with drainage provided to lower water table	< 2 m below grade
Presence of Hard or Energy-Absorbing Layer	No hard or soft layers	1. Hard surface layer: loosen prior to dynamic compaction. 2. Energy-absorbing surface layer: remove or stabilize with aggregate	Energy absorbing layer that limits depth of improvement, such as Zone 3 soil of 1 m or more in thickness at a depth that is impractical to remove
3. Determine Design Requirements			
Settlement	< 0.3 to 0.6 m for embankments	> 0.3 to 0.6 m if site conditions preclude large differential settlements	Settlement > design engineer can tolerate
Minimum Soil Property	Can usually achieve relatively high SPT, CPT, and PMT	May need wick drains in saturated Zone 2 soils to facilitate drainage	——
Depth of Improvement Limitation	Deposit < 9 m thick	Special equipment required for deposits in range of 9 to 12 m	Soils cannot be significantly Improved below 11 m
4. Estimate Costs			
Dynamic Compaction	Generally least expensive form of site improvement	Multiple phases could slightly increase cost	If costs exceed alternate forms of site
Surface Stabilization	Frequently not required	——	1 m layer could cost more than dynamic compaction

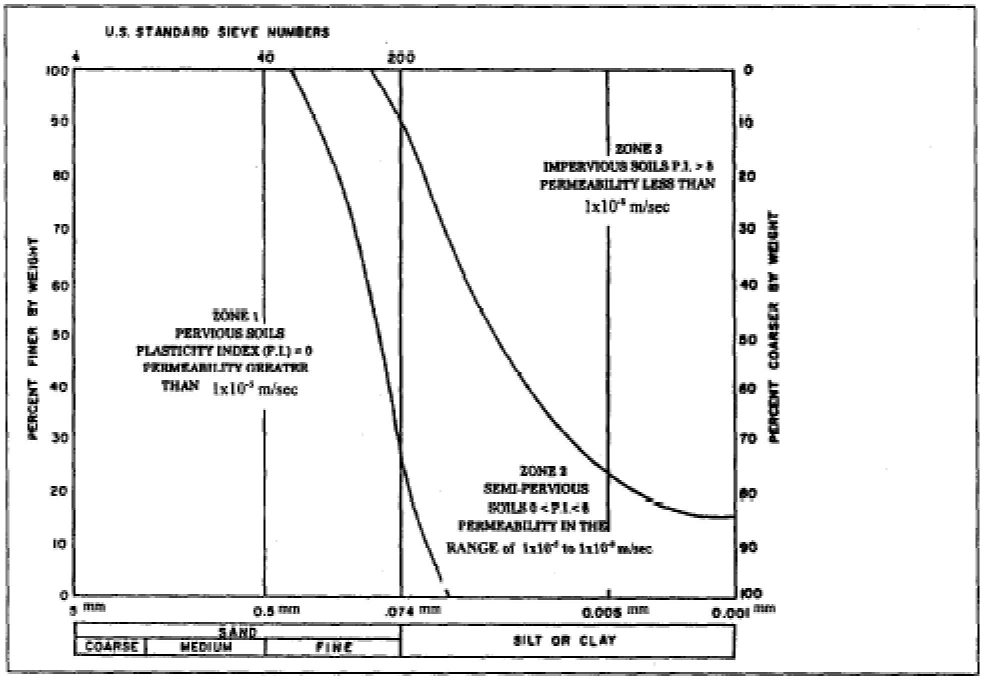


Figure 3.3 Grain Size Zones for Deep Dynamic Compaction Suitability

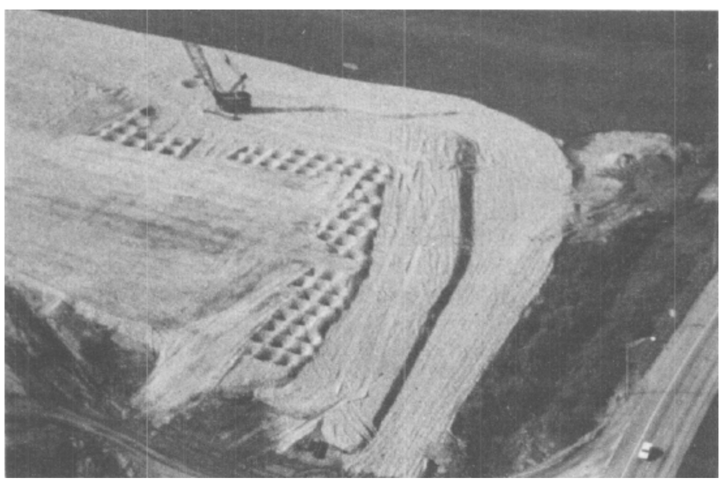


Figure 3.4 Deep Dyanmic Compaction of Closed Landfill Site (FHWA, 1995)

water content of the deposit is lowered. Generally, clayey soils have permeabilities of less than 10^{-8} to 10^{-9} m/s, so dissipation of excess pore water pressures generated during dynamic compaction cannot occur, except perhaps over a lengthy period of time. This makes dynamic compaction impractical for these deposits. Furthermore, the degree of improvement is generally minor.

Some improvements have been achieved in clayey fill deposits that are only partially saturated. This includes fills elevated well above the water level and with good surface drainage. In this case, improvement occurs as the particles are compacted before the deposits become fully saturated. After saturation occurs, no further improvement will be realized regardless of the amount of energy applied. Generally, the water content of the clayey soils prior to dynamic compaction should be less than the plastic limit of the deposit.

Intermediate Soil Deposits - Zone 2

There is a third zone of soils, labeled Zone 2 on Figure 3.3, that is intermediate between the most favorable soils and the unfavorable soils for dynamic compaction. Silts, clayey silts, and sandy silts fall into this category. Normally, the soils in Zone 2 have a permeability on the order of 10^{-5} to 10^{-8} m/s. Dynamic compaction works in these deposits, but because of the lower than desired permeability, the energy must be applied using multiple phases or multiple passes. Sufficient time should be allowed between the phases or passes to allow excess pore water pressures to dissipate. Sometimes, the excess pore water pressure takes days to weeks to dissipate. On some projects, wick drains have been installed in these formations to facilitate drainage.

ASSESS SITE RESTRAINTS

Site restraints may necessitate an alteration in the dynamic compaction procedure or supplemental construction activity to compensate for a site's deficiency. These site restraints should be evaluated in the preliminary study to determine what effect they might have on the project cost and timing.

Ground Vibrations

When a tamper strikes the ground, vibrations are transmitted off site. The vibrations are largest when heavier tampers and higher drop heights are used. If dynamic compaction is undertaken in a congested area, some off-site structures could be affected by the ground vibrations.

The U S Bureau of Mines (1980) has studied the effect of ground vibrations on structures and has established threshold particle velocities beyond which cracking in walls of homes may occur. These limits are shown in Figure 3.5. Numerous measurements from dynamic compaction projects have indicated that the frequency of ground vibrations from dynamic compaction is in the range of 6 to 10 hz. At this frequency, the U. S. Bureau of Mines criteria indicates that the particle velocities should be less than 13 and 19 mm/set for older and more modern construction to prevent cracks in the walls. Structural damage does not occur until the particle velocities exceed about 50 mm/set, although the tolerance to vibrations depends upon the condition of the structure.

Particle velocities can be measured with a portable field seismograph and compared with the criteria shown in Figure 3.6. Readings should be taken on the ground adjacent to the concerned facility.

The particle velocities that will develop as a result of dynamic compaction should be predicted in advance of construction to determine if threshold vibration levels will be exceeded. Figure 3.6 has been developed from measurements taken on numerous projects and can be used to predict particle velocities. The scaled energy factor incorporates the energy imparted into the ground from a single drop plus the distance from the point of impact to the point of concern. The chart is entered with the calculated scaled energy factor and a line projected vertically to the

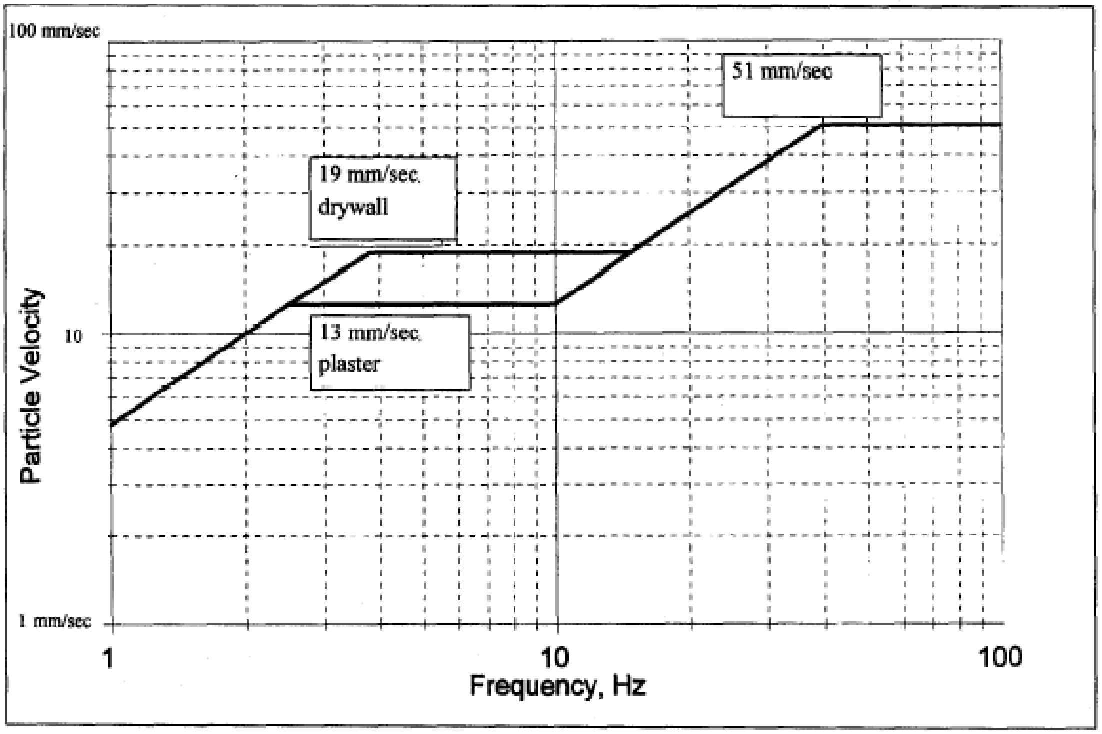


Figure 3.5 Damaging Particle Velocities

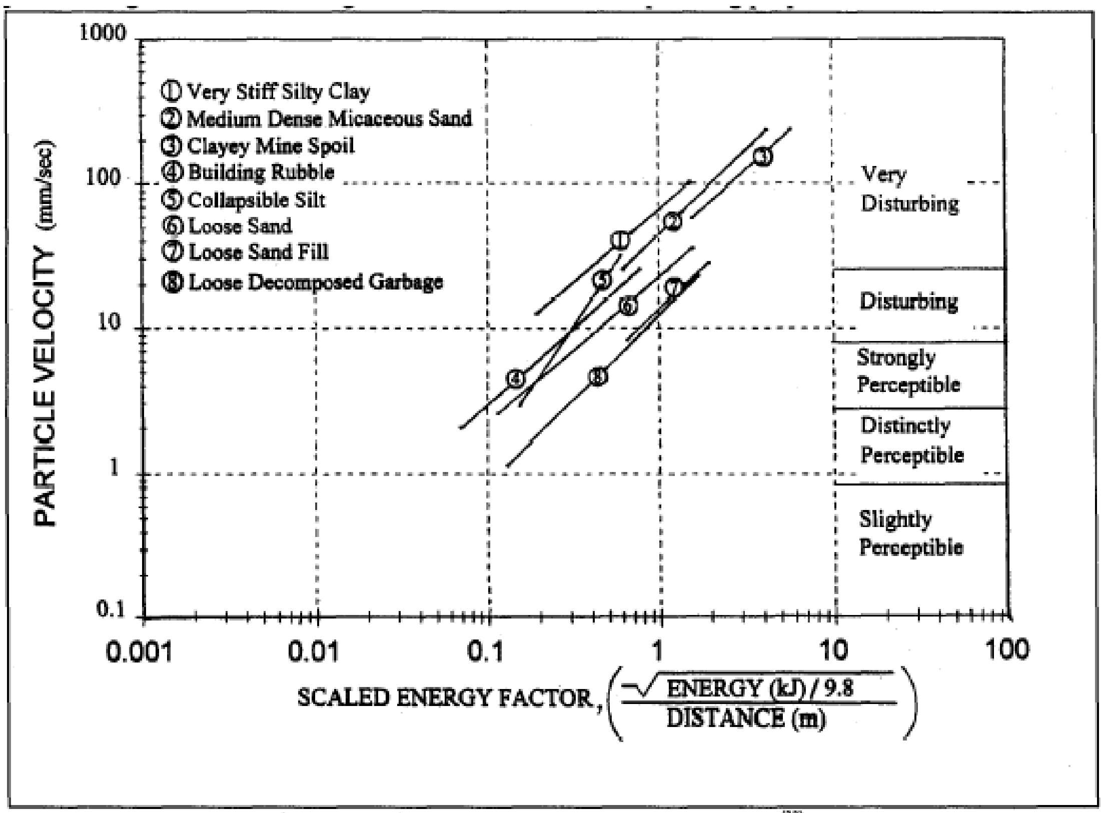


Figure 3.6 Relationship Between Particle Velocity and Scaled Energy Factor for Different Soils.

most appropriate soil type. A horizontal line is then extended laterally and the predicted particle velocity read off the vertical axis. This chart is based on records taken from many sites and provides a good estimation of ground vibration levels for planning purposes.

If dynamic compaction must be performed near an existing facility and the ground vibrations need to be minimized, some success has been obtained with digging a trench to a depth of approximately 3.0 m between the point of impact and the structure of concern. The trench should be installed at a location where it will not undermine the foundations of the structure or lateral support of a buried utility. An open trench is the most effective in reducing vibrations.

However, open trenches which could cause undermining or other concerns should be filled with some loosely placed soil or compressible material. The purpose of the trench is to cut off the Rayleigh wave, which is a surface wave that travels off site from the point of impact. At some sites, off-site ground vibrations have been reduced by reducing the thickness of the loose deposit by excavation and then using a lighter tamper and smaller drop height to density the remaining soils. Afterwards, the upper portion of the excavated soil can be replaced and densified in a similar manner.

Lateral Ground Displacements

Some lateral displacements occur in the ground following the impact. Unfortunately an established procedure has not been developed to predict lateral ground movements. Reliance is placed on experience and measured data reported in the literature. As part of the FHWA (1995) study on dynamic compaction three project sites were instrumented with inclinometers located at distances of 3.0 m and 6.1 m from the point of impact. Lateral ground displacements were measured at both of these locations. At a distance of 3.0 m from the point of impact, lateral displacements ranging from 152 to 3 18 mm were measured within the zone of 6.1 m below grade. At 6.1 m from the point of impact, the lateral ground displacements were only on the order of 19 to 76 mm within the upper 6.1 m of the soil mass. Less displacement would occur for sites where a smaller tamper and reduced drop height were used.

If there are roadways or buried utilities located close to the point of impact, the likelihood of permanent ground displacements should be considered. Field measurements of lateral displacement or ground vibrations can be used to assess potential damage at structure locations.

Particle velocity measurements have been made with a seismograph on the ground over buried utilities. (Wiss, 1981) Particle velocities of 76 mm/set have not damaged pipes and mains. Pressure pipelines have withstood 250 to 500 mm/set without distress.

High Water Table

Water table levels within approximately 2 m below the level of dynamic compaction often cause problems. During impacting, crater depths are frequently on the order of 0.6 to 1.2 m, and high pore water pressures generated in the soil mass generally cause the ground water table to rise. This could result in water filling into the craters. Additional drops could cause intermixing of the soil and water with subsequent softening of the upper portion of the soil mass. If the water table is within 2 m of ground surface, consider:

- Lowering the ground water table by dewatering ditches or dewatering wells.
- Raising the ground surface by placing fill.

ENGINEERING

Dynamic Compaction is the dropping of a heavy weight as shown in Figure 3.7 to compact deep layers. Application of high energy by repeated raising and dropping a tamper with a mass ranging from 5 to 18 Mg from drop heights of 9 to 30 m. A conventional crane is used. Depth of improvement ranges from 3 to 11 m. Suitable primarily for granular soils.

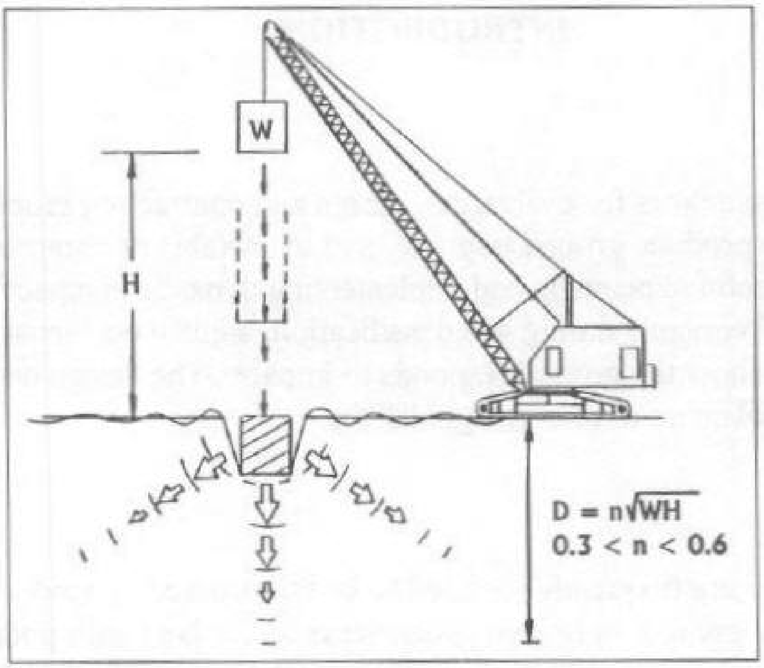


Figure 3.7 Schematic of Deep Dynamic Compaction

Application: (1) Densification of loose deposits (2) Collapse of large voids (sink holes)

Advantages: (1) Compacts soils at depth (deeper than conventional compaction methods)
(2) Each pass should result in shallower craters, showing improvement
(3) OK for heterogeneous deposits and mixtures. Also uncompactable building rubble
(4) Densification makes bearing layers more uniform – less differential settlement
(5) Densification of layers below ground water table – no dewatering required
(6) Usually non-specialty contractors can perform

Disadvantages: (1) Ground vibrations affect property
(2) Ground water table >2 m otherwise pumping or softening

(3) Very loose deposits i.e., old landfill requires surface layer of crushed stone or sand for confinement thickness 0.3 to 0.9 m.

Soil Type: See Figure 3.3 (ground water table should be deeper than 2 m from surface)

(1) Zone 1 (best) $k > 10^{-5}$ m/s

(2) Zone 2 needs intermediate "rest" multiple passes for pore water pressure dissipation

(3) Zone 3 (poor) clays $k < 10^{-8}$ m/s

Vibrations: See Figures 3.5 and 3.6. Damage at 50 mm/s, minor at 10 mm/s, annoying at 2.5 mm/s

Trenches have been used to dampen surface vibration

Tampers 15 to 30 Mg should not be used within 6 – 7 meters from buried structures.

Equipment: Tampers: 5 to 27 Mg usually solid steel or steel casing filled with concrete

Shape: round preferred – tamper rotates as lifted – guy wires used

Contact pressures: W/Area ≈ 36 to 72 kPa

Drop heights: 9 to 30 m (but 12 m is common)

Spacing: up to 14 m (usually 2 to 3 m)

Depth of improvement: up to 40 m (others say 12m)

Common: 16 Mg tamper (16 tons) drop height 25 m (90 ft) improves depth of 11m (36 ft)

Table 3.2 summarizes the design guidelines.

EXAMPLES

Calculate Ground Vibrations

DDC with 890 kN weight falling 15 m to compact a 9 m loose sand layer. If particle velocity is to be less than 5 mm/s, is a building 30 m away ok?

$$\text{Scaled Energy} = \frac{\sqrt{\text{Energy (kJ)}/9.8}}{\text{Distance (m)}}$$

$$\text{Scaled Energy} = \frac{\sqrt{(890\,\text{kN} \times 15\,\text{m})/9.8}}{(30\,\text{m})}$$

$$\text{Scaled Energy} = 1.23$$

From Figure 3.6, particle velocity would be ~12 mm/sec. NOT APPROVED

Table 3.2 Design Guidelines (from FHWA, 1995)

Parameters to be Determined	Evaluation Process
Step 1: Selection of tamper and drop height for required depth of improvement Equation 1: $D = n(WH)^{0.5}$	A. Determine thickness of loose deposit from subsurface exploration or the portion of the deposit that needs densification to satisfy design requirements. B. Use Equation 1 and select n value from Table 3.3 for soil type. n varies between 0.3-0.8 (Figure 3.8) C. Use Figure 3.9 as a guide in selecting tamper mass and drop height for dynamic compaction equipment currently is use.
Step 2: Determine applied energy to achieve required depth of improvement	A. Use Table 3.4 to select the unit energy for the proper deposit classification. B. Multiply the unit energy by the deposit thickness to obtain the average energy to apply at ground surface.
Step 3: Project area to densify	A. For level sites, use a grid spacing throughout the area in need of improvement plus a distance beyond the project boundaries equal to the depth of improvement. B. If slope stability is a concern, improvement over a wider plan area may be required. C. At load concentration areas, apply additional energy as needed.
Step 4: Grid spacing and drops Equation 2: $AE = \dfrac{N(W)(H)(P)}{(\text{grid spacing})^2}$ where: N = number of drops P = number of passes W = mass of tamper H = drop height	A. Select a grid spacing ranging from 1.5 to 2.5 times the diameter of the tamper. B. Enter W and H from step 1 and applied energy from step 2 into Equation 2. C. Use Equation 2 to calculate the product of N and P. Generally 7 to 15 drops are made at each grid point. If the calculations indicate significantly more than 15 or less than 7 drops, adjust the grid spacing.
Step 5: Multiple Passes Prediction of crater depths or ground heave in advance of dynamic compaction is difficult. The contract should provide for multiple passes where very loose deposits like landfills are present or where silty deposits are nearly saturated.	A. Crater depths should be limited to the height of the tamper plus 0.3 m. B. Energy application should stop if ground heave occurs. C. If items A or B occur before the required number of drops are applied, multiple passes should be used to: • permit ground leveling if item A occurs • allow pore pressure dissipation if item B occurs
Step 6: Surface stabilizing layer	A. Not needed for Zone 1 soils. May be required for Zone 2 soils if nearly saturated. Usually required for landfills. B. When surface stabilizing layer is used, the thickness generally ranges from 0.3 to 0.9 m.

Table 3.3 Recommended n Values for Different Soil Types (from FHWA 1995)

Soil Type	Degree of Saturation	Recommended n Value*
Pervious Soil Deposits – Granular Soils	High	0.5
	Low	0.5 – 0.6
Semipervious Soil Deposits – Primarily silts with plasticity index of < 8	High	0.35 – 0.4
	Low	0.4 – 0.5
Impervious Deposits – Primarily clayey soils with plasticity index of > 8	High	Not recommended
	Low	0.35 – 0.40 Soils should be at water content less than the plastic limit.

* For an applied energy of 1 to 3 mJ/m^2 and for a tamper drop using a single cable with a free spool drum.

Table 3.4 Applied Energy Guidelines (from FHWA 1995)

Type of Deposit	Unit Applied Energy (kJ/m^3)	Percent Standard Proctor Energy
Pervious coarse-grained soil – Zone 1 of Figure 3.3	200-250	33-41
Semipervious fine-grained soils – Zone 2 and clay fills above the water table – Zone 3 of Figure 3.3	250-350	41-60
Landfills	600-1100	100-180

Note: Standard Proctor energy equals 600 kJ/m^3 .

Calculate Tamper Weight

What is the minimum weight required to compact a 10 m layer?

$$\text{Depth} = n\sqrt{\text{Weight} \times \text{Height}}$$

$$10\ \text{m} = 0.6\sqrt{\text{Weight} \times 15\ \text{m}}$$

$$\text{weight} = 18.5\ \text{Mg} = 180\ \text{kN}$$

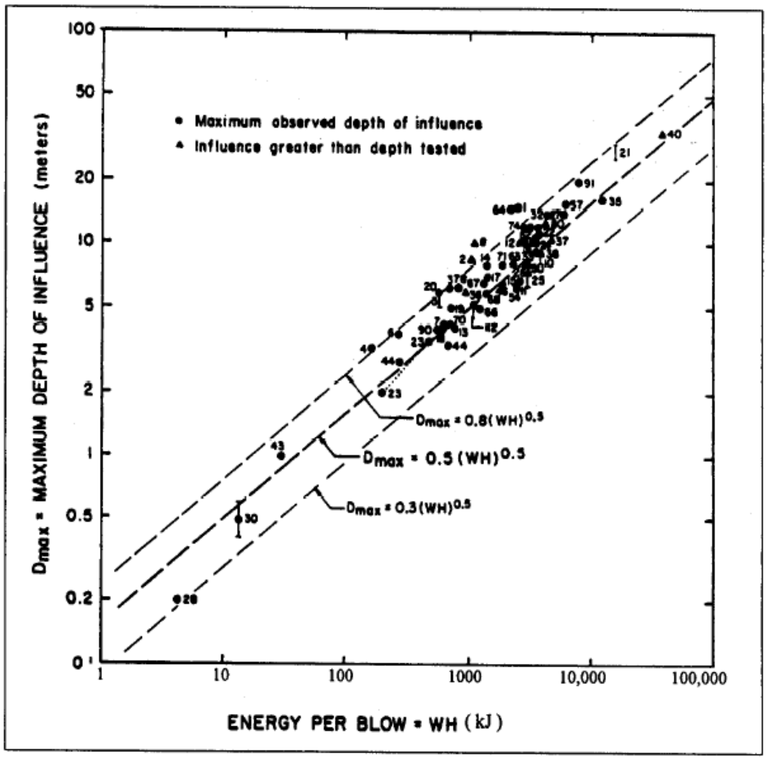


Figure 3.8 Trend Between Maximum Depth of Influence and Energy Per Blow
(from FHWA, 1995)

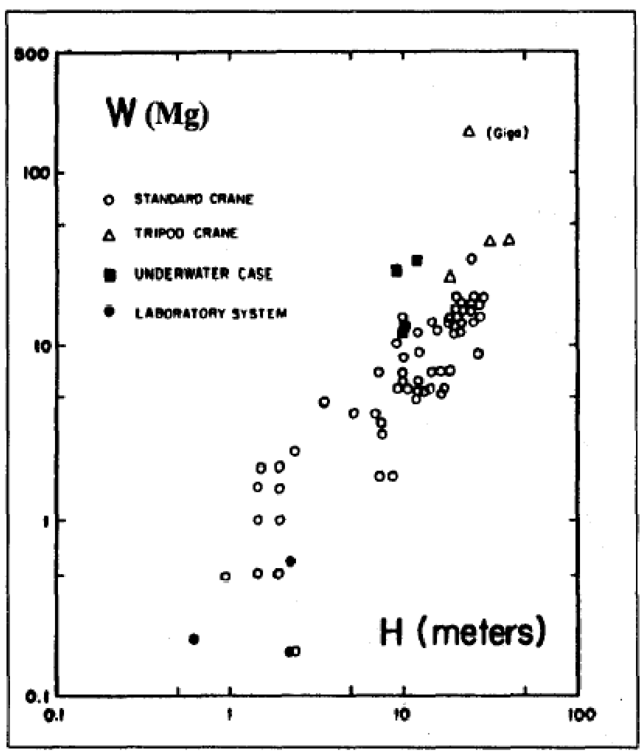


Figure 3.9 Relationship Between Size of Tamper and Drop Height (from FHWA, 1995)

Cost Estimate

Estimate the cost to densify a 7.5 Acre site with a 200kN tamper.

Use Table 3.5:

200 kN ~ 20 Mg

Cost per m^2 is approximately $13.35

7.5 Acres = 30,350 m^2

$13.35 x 30,350 m^2 = $405,172.50

Table 3.5 Dynamic Compaction Costs (from FHWA, 1995)

Size of Tamper Required (Mg)	**Unit Cost Dollars/m^2**
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 during 1985 to 1993.

Assess Deep Dynamic Compaction System

Determine the tamper weight, drop height, spacing for a 8 m thick deposit of a Zone 1 soil.

1) Zone 1 Soil thickness of 8m, find the Average Energy (AE) to be applied

   Table 3.4, AE ≈ 250 kJ/m^3 x 8m = 2 MJ/m^2

2) Use Depth to determine product of WH required:

   $$\text{Depth} = n\sqrt{\text{Weight} \times \text{Height}}$$

   say n = 0.5

   $$8 = 0.5\sqrt{WH}$$

   WH = 256

3) Determine tamper weight and drop height

   Figure 3.9 → Assume 10 Mg tamper (common size) → $H = \frac{256}{10} = 25.6\,\text{m}$ (high)

   Assume 15 Mg tamper → $H = \frac{256}{15} = 17\,\text{m}$ (better)

4) Determine tamper dimension and grid spacing

Assume contact pressure ≈ 40 kPa (typically 36 – 72 kPa)

$$\text{Area} = \frac{15\text{Mg} * \frac{10\text{kN}}{\text{Mg}}}{40\text{kPa}} = 3.75\,\text{m}^2 = \frac{\pi d^2}{4} \quad d = 2.19\,\text{m}$$

Since d of tamper equals 2.19 m, the 3.5 m grid pattern is ok (5d up to 14m rec'd)

5) Use Average Energy Equation to determine number of drops:

$$\text{Average Energy} = \frac{\text{Number of Drops}(\text{Tamper Weight})(\text{Drop Height})(\text{\# of Passes})}{(\text{grid spacing})^2}$$

$$\frac{2000\,\text{kJ}}{\text{m}^2} = \frac{N\left(15\,\text{Mg} * \frac{10\,\text{kN}}{\text{Mg}}\right)(17)(\text{say 1 pass})}{(3.5\text{m})^2}$$

N = 9.6 drops, say 10 drops per location, at 1 pass

6) Closest distance to structure with maximum particle velocity of 5 mm/s

$$\text{Scaled Energy} = \frac{\sqrt{\text{Energy}(kJ)/10}}{\text{Distance}(m)}$$

From Figure 3.6, Scaled Energy = 0.2

$$0.2 = \frac{\sqrt{\left(15\text{Mg} * \frac{10\,\text{kN}}{\text{Mg}}\right)(17\,\text{m})/10}}{\text{Distance}\,(\text{m})} \qquad \text{Distance} = 80\text{m}$$

SPECIFICATIONS

Since much of Deep Dynamic Compaction is empirical and site based, often the contracting and specifications need to be well defined. Primarily, there are two cases, a method specification and a performance specification. In a method spec, the design is done in house and prescribed to the contractor. In a performance spec, only the desired outcome is required of the contractor. They are then free to determine the course of treatment to achieve owners minimum specifications. Table 3.6 compares and contrasts the two specifications.

Table 3.6 Contracting for Dynamic Compaction (from FHWA,1995)

Method Specification	Performance Specification
Agency should have in-house experience or hire a consultant with experience to prepare detailed specifications for contractors. Specifications should include: ~ 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 work area	Owner or designer prepares specification outlining desired end product. This could include: ~ Minimum property values ~ Maximum permissible settlement ~ Other objectives of site improvement Owner provides initial subsurface data and lateral extent of project site.
Owner or designer provides: ~ Subsurface investigation data ~ Monitoring during construction ~ Borings and tests after dynamic compaction	The contractor is required to meet the minimum specified end product and is responsible for: ~ Proper equipment and work plan ~ Meeting project deadlines ~ Safety ~ Field monitoring ~ Additional subsurface exploration as required to properly prepare dynamic compaction plan ~ Verification of end product
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 of designer	To obtain a quality work product, the designer should require: ~ Only experienced dynamic compaction contractors to bid ~ Submittal of work plan for review and comment ~ A method for adjusting differences of opinion between designer and contractor

CASE HISTORY DENSIFICATION OF LOOSE POCKETS AND VOIDS (FROM FHWA, 1995)

Introduction

A three-story structure was planned over an 8000 m^2 site in Florida. The structural loads were relatively light; but the initial subsurface exploration indicated the presence of sinkholes and voids due to dissolution of the limestone formations. In addition, there was a large amount of heterogeneity in the subsurface profile throughout the site, which led to large predicted differential settlements.

A typical boring log is shown in Figure 3.10. The predominant soil type is a silty fine sand grading to a fine sand with seams of sandy clay. The low SPT values are indicative of either a void or a soil that has collapsed into a void. Other soil borings that are not shown indicate a relatively dense soil profile especially where the calcareous materials within the silty sand have caused some cementation. Thus, the foundation support would range from very good load support on the cemented materials to very poor load support in the cavernous areas.

The initial soil profile led to settlement predictions ranging from 23 mm to 74 mm assuming no large collapse of voids. The resulting 51 mm differential settlement was considered too large for the structure to tolerate. In addition, the presence of a cavity a short distance below foundation level would result in a very risky design.

The designer indicated that shallow foundations could be used for this project provided the soils were made more homogeneous as far as load support and no voids were present within the depth range of 7.6 m to 9.1 m below ground surface.

Dynamic Compaction Considerations

The soils at this site are predominantly a silty sand formation that would place them into the Zone 2 category according to Figure 3.3. This means that the soils would be suitable for dynamic compaction, but that multiple phases and/or passes would need to be made throughout the area since the generation of pore water pressures takes time to dissipate.

For a depth of improvement of 7.6 m, the use of equation 1 and an empirical n value of 0.4, the energy per blow (W. computes to 3.56 MJ). The local contractor doing dynamic compaction had a 15 Mg tamper, available and for this size tamper the required drop height computes to be 24 m.

Using Table 3.4 for applied energy requirements as a guide, the average applied energy would calculate to be approximately 300 kJ/m3 multiplied by the required depth of improvement of 7.6 m, resulting in an average applied energy at the surface of 2.28 MJ/m*. This energy should be applied with two phases and two passes per phase to allow pore water pressures to dissipate between each pass. Because of the possibility of voids or caverns at any location, additional energy might need to be applied where large ground depressions would occur.

The maximum degree of improvement following dynamic compaction would be an SPT value on the order of 35 and a maximum limit pressure of a pressuremeter test of 1.4 to 1.9 MPa. These are upper-bound values, and the degree of improvement would be less than this depending on the amount of energy applied.

Actual Project Records

The site improvement was undertaken using a performance specification with a specialty contractor. The contractor selected a 15 Mg tamper and a drop height of 20 m. The energy was applied in 2 phases with 3 passes in the first phase and 2 passes in the second phase. Additional drops were made at sinkhole locations. The energy application is summarized in Table 3.7. The average energy application was 1.6 MJ/m*. The induced ground compression calculated to be 9.1 percent of the anticipated depth of improvement of 7.6 m.

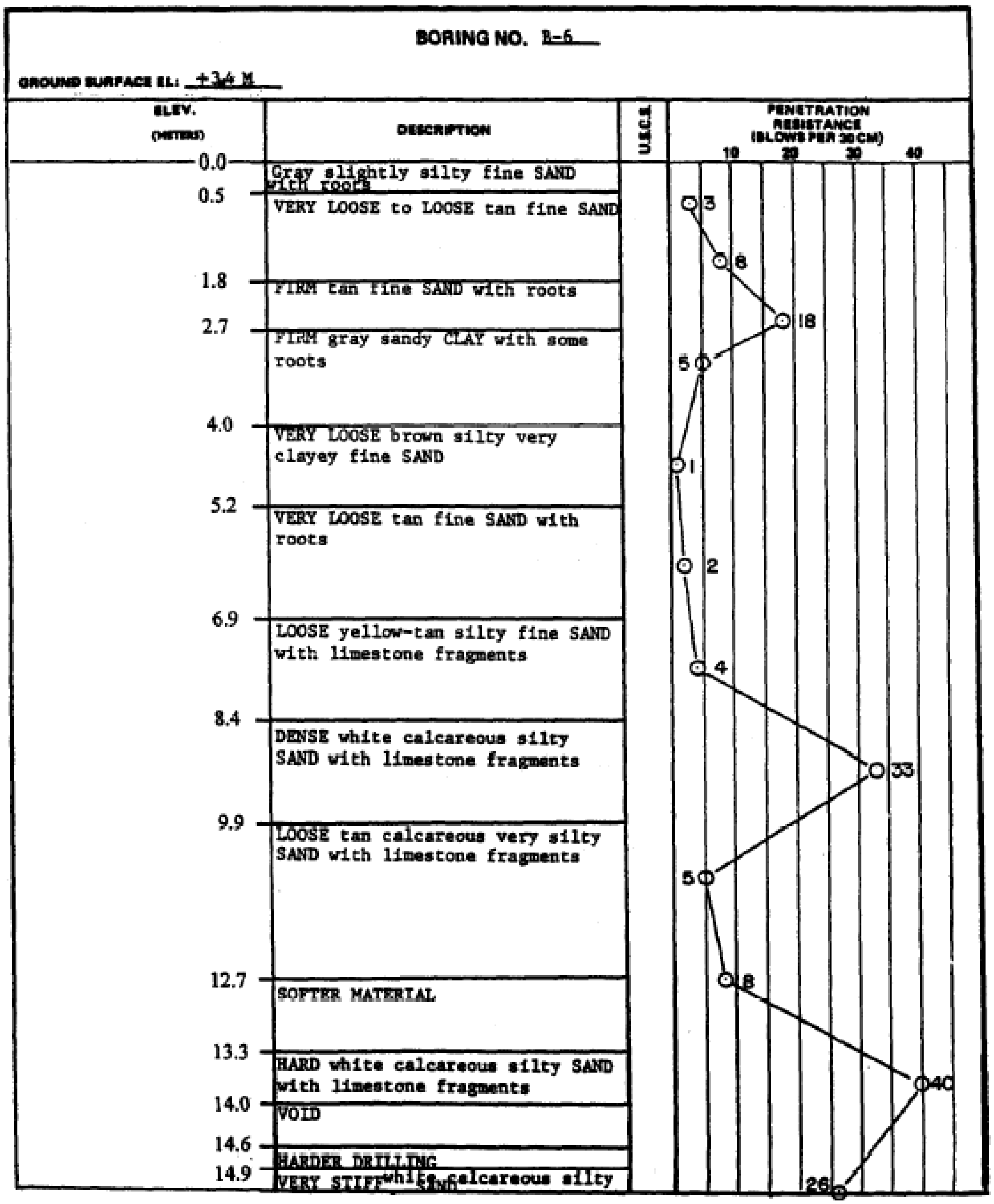


Figure 3.10 Soil Profile (from FHWA, 1995)

A comparison of average SPT values taken before and after dynamic compaction is shown in Figure 3.11. Although some improvement occurred in the standard penetration resistance values, the improvement is still less than one would predict. The specialty contractor felt the SPT values were somewhat misleading for this project. For this reason, pressuremeter tests were also performed before and after dynamic compaction. Figure 3.12 indicates the

Table 3.7 Dynamic Compaction Records

Phase	Pass	Grid (m)	Location	Blows/ print	Energy (kJ/m^2)	Induced Settlement (mm)
1	1	9.1 × 9.1	Primary	8	281	121
1	2	9.1 × 9.1	Primary	9	317	98
1	3	9.1 × 9.1	Primary	9	317	70
2	1	9.1 × 9.1	Intermediate	9	317	97
2	2	9.1 × 9.1	Intermediate	10	352	93
Ironing		Over-lapping	Continuous	1	35	89
7		Void	At observed sink hole locations	10	---	
Total				Total of 8895 blows	1,619	694

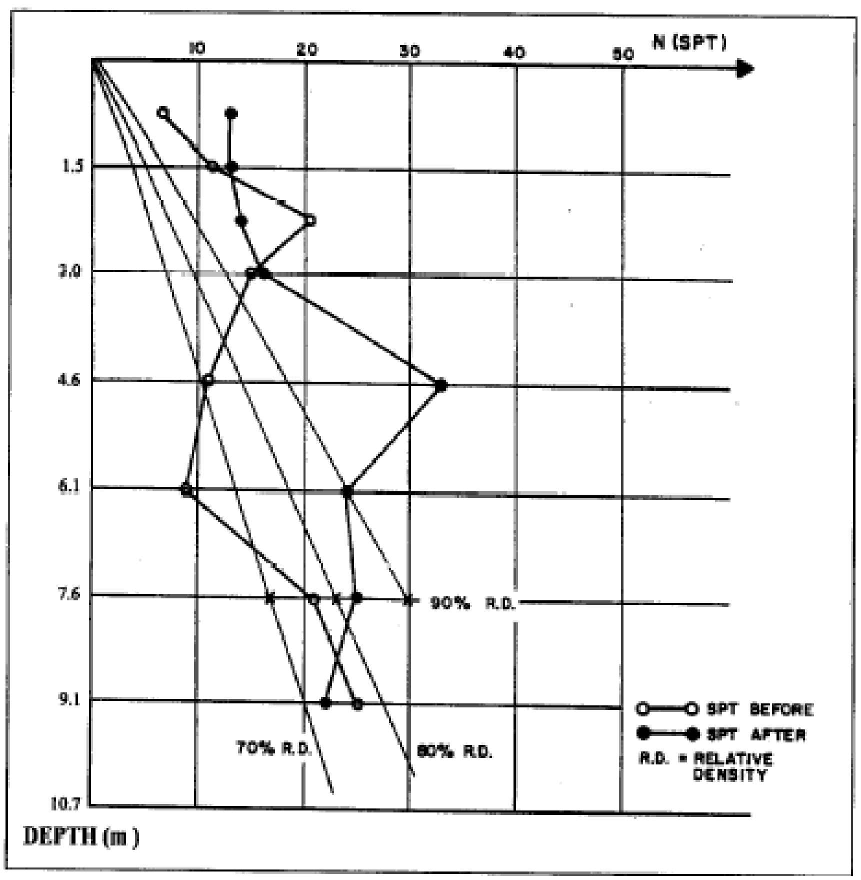


Figure 3.11 SPT Before and After Deep DynamicCompaction

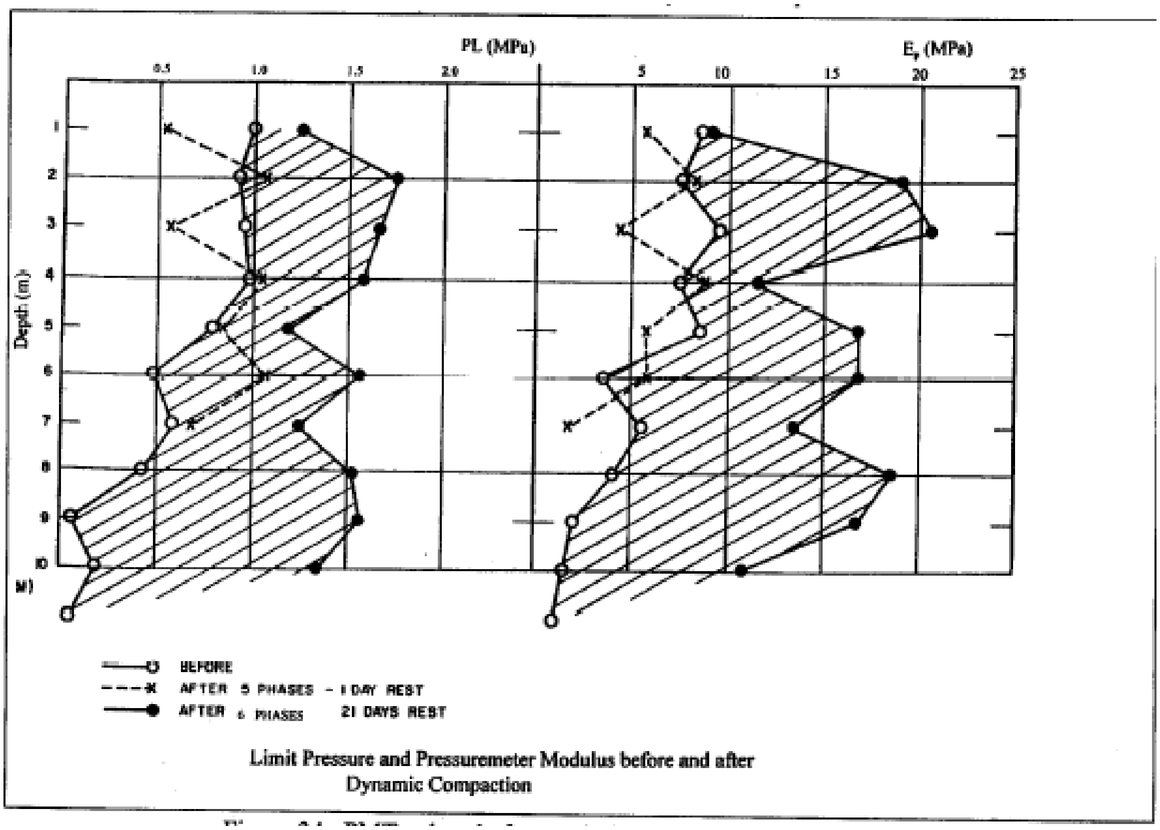


Figure 3.12 PMT Limit Pressures Before and After Deep Dynamic Compaction

average limit pressure and pressuremeter modulus values before and at two time intervals after completion of the dynamic compaction. The limit pressure and the modulus show a relatively uniform degree of improvement with depth, which was one of the desired results, and the limit pressure is also in accordance with the predicted value.

The pressuremeter test results performed at various intervals of time after dynamic compaction illustrate the improvement that takes place well after the energy has been applied. During dynamic compaction, settlements were taken on a grid basis throughout the project site. Figure 3.13 shows the induced settlement contours following the first three phases of dynamic compaction. These contours indicate that there are two locations where the settlement is much greater than normal. This would correspond to approximate column locations K-6 and C-6. The greater settlement in these areas indicates the presence of cavities or very loose deposits. For this reason, additional energy was applied in these areas.

Important Conclusions from This Project

- The depth of improvement of 7.6 m was reached even though the energy per blow was slightly less than recommended by equation 1.
- The energy that the contractor used for densification was slightly less than suggested by Table 3.4. The pressuremeter test shows good improvement was reached, but the SPT

values show that there could have been more improvement if additional energy had been applied.

- Plotting of the settlement pattern following different phases of energy application was very helpful in determining where cavities or sinkholes were present. In these areas, additional energy was applied.
- The increase in pressuremeter properties with time is clearly demonstrated by Figure 3.12. This phenomena of strength increase following rest periods has been measured at many sites ranging from sandy soils to fine grain soils. Borings with tests made during dynamic compaction or immediately thereafter will therefore not measure the total improvement.

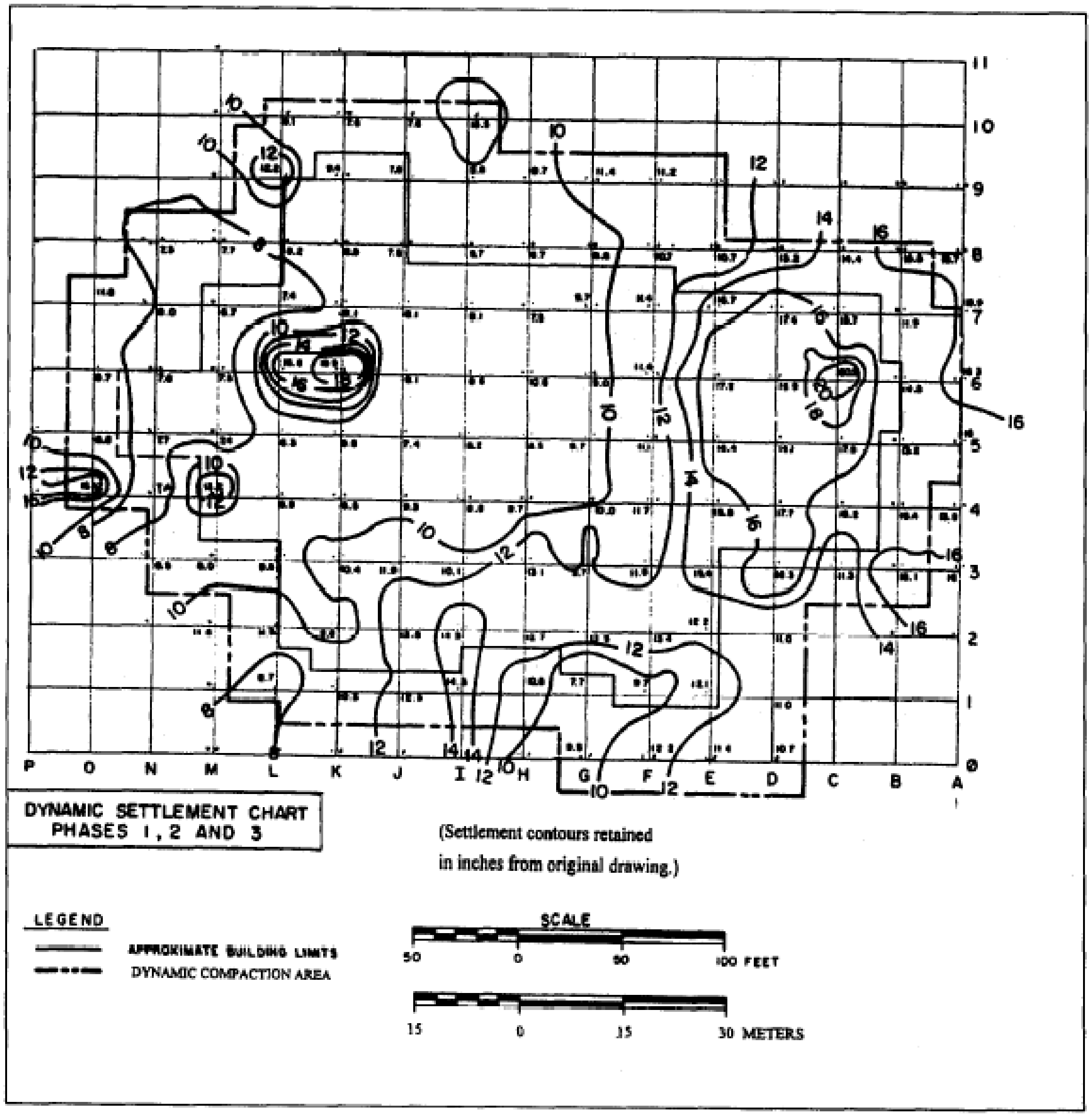


Figure 3.13 Plan of Induced Settlement Contours

REFERENCES

Drumheller, J.C., *Dynamic Compaction and Highway Construction*, *Case Histories/Summaries of Dynamic Compaction Projects for Highway Construction in the U.S.*, 1970-1991, Transportation Research Board, February 1992.

FHWA (1999) Ground Improvement Technical Summaries Demonstration Project 116, Vol. 1 and 2, Publication No. FHWA-SA-98-086, Authors: Elais, V. Welsh, J., Warren, J., and Lukas, R.

FHWA (1995) Geotechnical Circular No. 1 Dynamic Compaction, Publication No. FHWA-SA-95-037, Author: Lukas, R.G.

FHWA (1986) Dynamic Compaction for Highway Construction Vol 1. Design and Construction Guidelines" FHWA RD-86/133. Author: Lukas, R.G.

USBM (1980) "Structure Response and Damage Produced by Ground Vibration from Surface Mine Blasting," USBM RI 8507 Authors: Siskind, D.E., Stagg, M.S., Kopp, J.W., and Dowding, C. H.

Wiss, J.F. "Construction Vibrations: State of the Art," ASCE Geotechnical Engineering Division, Vol. 107, GT2, pp. 167-181, February 1981.

Chapter 4

PREFABRICATED VERTICAL DRAINS

The benefits of preloading include; (1) an increase in bearing capacity by reducing excess pore pressures, and (2) the reduction of the compressibility of weak ground by accelerating consolidation. The concept is to apply a vertical load (surcharge greater than the anticipated foundation load), allow the layer to consolidate, remove the surcharge and apply foundation load. Additionally, vertical drains are used to accelerate consolidation by shortening drainage path. Figure 4.1 shows the concepts of preloading and preloading with vertical drainage.

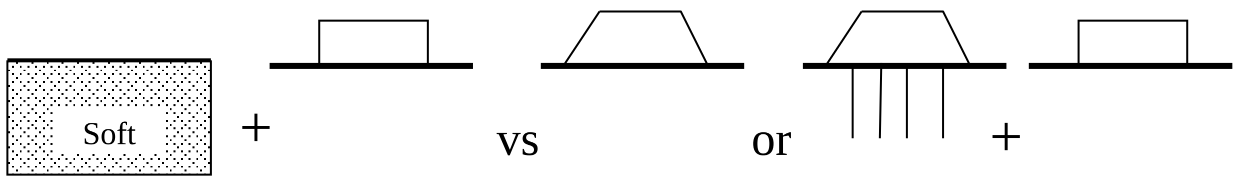


Figure 4.1 Comparison of Self-Weight Consolidation, Surcharging, and Surcharging with Vertical Drains

PRELOADING

Preloading or surcharging is used to accelerate the reduction of excess pore pressures or consolidation of a weak deposit by placing a load larger tan the permanent foundation load for a predetermined period of time. Figure 4.2 shows schematically how a preload is placed. The implications of the preloading on consolidation behavior is shown in Figure 4.3.

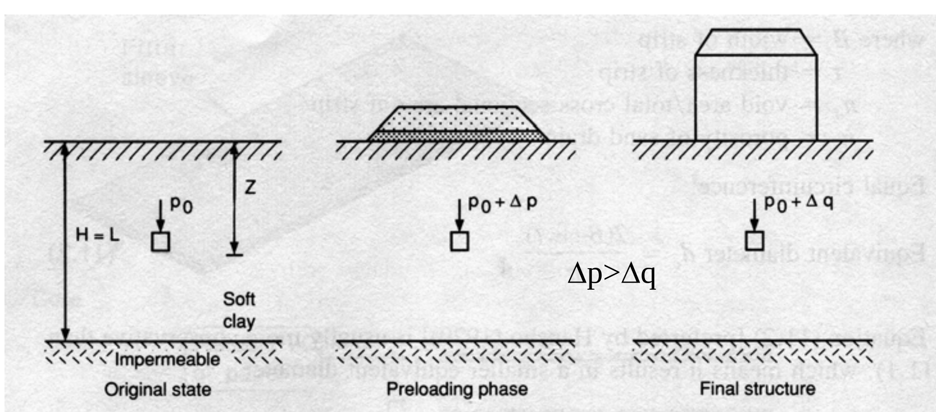


Figure 4.2 Schematic of Surcharging (from Hausmann, 1990)

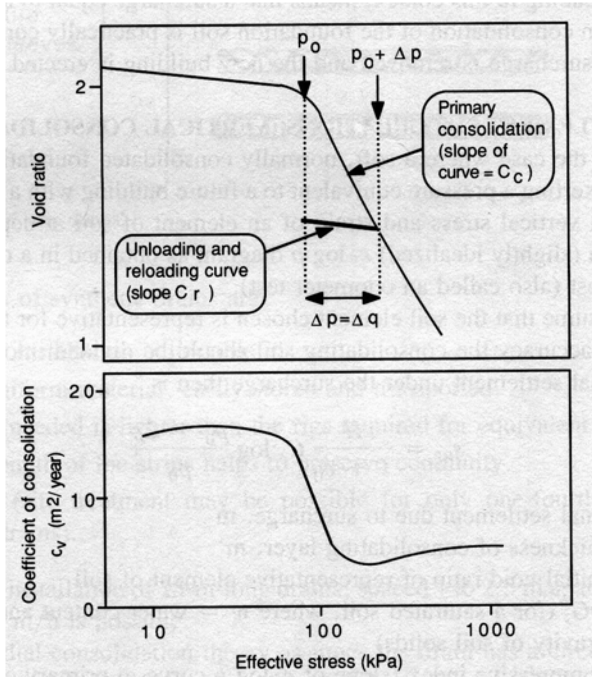


Figure 4.3 Implications of Surcharging and Traditional Consolidation Curve

Since the benefits of preloading are intrinsically related to time-rate of consolidation, it is useful to consider Figure 4.4. The magnitude of settlement is shown vs. time and surcharge amount. The foundation load, ΔP_f , will cause a settlement, δ_{fF} , requiring time, t_1, to achieve the total settlement. However, application of a surcharge + foundation loading, $\Delta P_F + \Delta P_S$, will achieve the same total settlement, δ_F, at time t_2, which is significantly less than time t_1. Hence by applying the surcharge + foundation load for a time t_2, and then removing the surcharge, the foundation load, ΔP_F could be applied without any significant additional settlement.

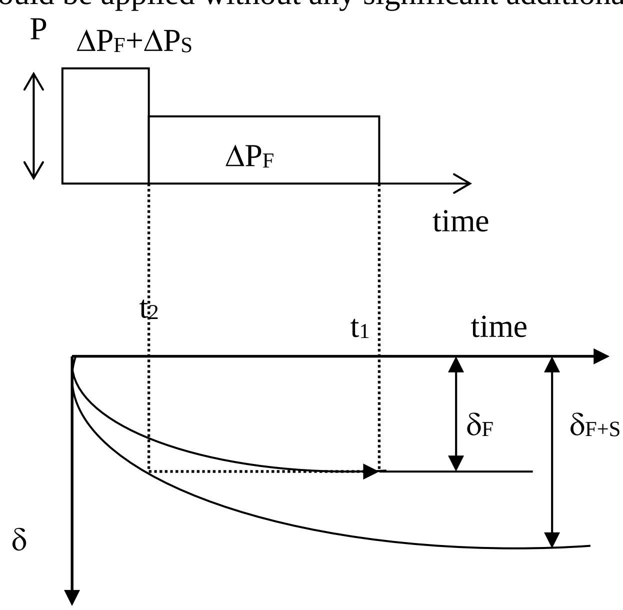


Figure 4.4 Time-Rate of Consolidation with Surcharging

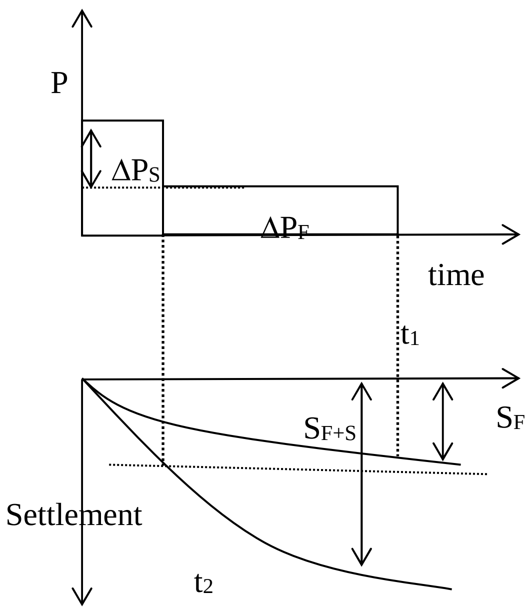
Rowe (1968) suggests only relatively impermeable soils cv < $3x10^{-7}$ m/s will benefit from the use of vertical drains.

Originally sand drains were used for vertical drainage; however, now filter fabric and plastic (100mm @ 2 to 6mm) drains are more commonly used. Key requirements for these drains are:

1. $k_w > k_{soil}$
2. filter should not clog
3. fabric should not seal plastic channels
4. wick should withstand biological and chemical attack

PRELOADING CALCULATIONS

When calculating the amount of surcharge preload necessary to eliminate the appropriate amount of settlement in a set period of time, the governing equation is still Terzaghi's one dimensional consolidation equation. The calculation steps are outlined in Figure 4.5



1) Let ΔP_F be the stress applied by building foundation
2) ΔP_s = the surcharge in addition to building foundation
3) S_F = Settlement of foundation, no surcharge
4) S_{F+S} = Settlement of foundation + surcharge.

Figure 4.5 Settlement vs. Time and Surcharge Loading

Preloading Consolidation Theory

Settlement due to applied foundation load:

$$S_F = \left(\frac{H}{1+e_0}\right)\left[c_c \log \frac{P_0' + \Delta P_F}{P_o'}\right]$$

Settlement of combined surcharge and foundation load:

$$S_{F+S} = \left(\frac{H}{1+e_0}\right)\left[c_c \log\frac{P_0' + \left[\Delta P_S + \Delta P_F\right]}{P_o'}\right]$$

The degree of consolidation at time t_2 after load application (sufficient to eliminate all settlement) can be expressed as:

$$U = \frac{S_F}{S_{F+S}} = \frac{\log\dfrac{P_o' + \Delta P_F}{P_0'}}{\log\dfrac{P_0' + \Delta P_F + \Delta P_S}{P_0'}}$$

The time required to achieve this percentage of consolidation is a function of the time factor T

$$T = \frac{\pi}{4}U^2 \quad U \le 60\% \text{ or } T = 1.781 - 0.933\log(100 - U) \quad U > 60\%$$

$$t_2 = \frac{T\left(\dfrac{H}{N}\right)^2}{c_v}$$

Then, how much surcharge ΔP_S has to be applied for a given time t to produce X% consolidation under foundation load ΔP_F alone?

1) Calculate S_F and S_{F+S}
2) S_{F+S} @ t = $XS_F/100$
3) U = SF+S @ t / SF+S
4) Find T
5) $t = T(H/N)^2/c_v$

Preloading Example

A highway bridge will cause a permanent ΔP_F = 115 kN/m^2. What surcharge ΔP_S is required to eliminate total bridge settlement in 9 months for a double drained case?

Given: H = 6 m, c_c = 0.28, e_0 = 0.9, c_v = 0.36 m^2/month, P_0' = 210 kN/m^2, N=2

1) $$S_F = \left(\frac{H}{1+e_0}\right)\left[c_c \log\frac{P_0' + \Delta P_F}{P_o'}\right] = \left(\frac{6}{1+0.9}\right)\left[0.28\log\frac{210+115}{210}\right] = 0.1667\text{ m}$$

2) $t = \dfrac{T\left(\frac{H}{N}\right)^2}{c_v} \therefore T = \dfrac{c_v t}{\left(\frac{H}{N}\right)^2} = \dfrac{0.36 \times 9}{\left(\frac{6}{2}\right)^2} = 0.36$

$$T = \frac{\pi}{4}U^2 \quad U = \sqrt{\frac{4T}{\pi}} = \sqrt{\frac{4(0.36)}{\pi}} = 67.7\% > 60\%$$

$$T = 1.781 - 0.933\log(100 - U) \; 100 - U = 10^{\frac{1.781-T}{0.933}} = 10^{\frac{1.781-0.36}{0.933}} = 33.346 \quad U = 66.65\%$$

3) But $U = \dfrac{\log\dfrac{P_o' + \Delta P_F}{P_0'}}{\log\dfrac{P_0' + \Delta P_F + \Delta P_S}{P_0'}} = \dfrac{\log\dfrac{210+115}{210}}{\log\dfrac{210+115+\Delta P_S}{210}} = \dfrac{0.1897}{\log\dfrac{210+115+\Delta P_S}{210}}$

$$\log\left[\frac{(325 + \Delta P_S)}{210}\right] = \frac{0.1897}{0.6665} = 0.2846$$

$$\frac{(325 + \Delta P_S)}{210} = 10^{0.2486} = 1.9256$$

$$\Delta P_S = 1.9256(210) - 325 = 79.4 \text{ kN/m}^2$$

4) So surcharge is 115 + 79.4 = 194.4 kN/m^2

Check $S_F = \left(\dfrac{H}{1+e_0}\right)\left[c_c \log\dfrac{P_0' + \Delta P_F}{P_o'}\right] = \left(\dfrac{6}{1+0.9}\right)\left[0.28\log\dfrac{210+194.4}{210}\right] = 0.25$

$$U = \frac{S_F}{S_{F+S}} = \frac{0.1667}{0.25} = 66.65\%$$

5) If $\gamma_{surcharge}$ is approximately 20 kN/m^3, then $H_{surcharge}$ = 194.4/20 = 9.72 m.

Preloading with Radial Drainage

Vertical drains are often used in conjunction with preloading to accelerate the consolidation. Conceptually, the amount of settlement will remain the same, but it will occur under a shorter period of time since the drainage path is greatly reduced. Figure 4.6 illustrates the addition of vertical drains to the preloading concept.

Traditionally, sand drains were used for vertical drainage. In recent years, more efficient prefabricated drains called "wick drains" are used. Since the original mathematical development was done for sand drains, it will be presented in the next section. The same governing equations are appropriate for wick drains.

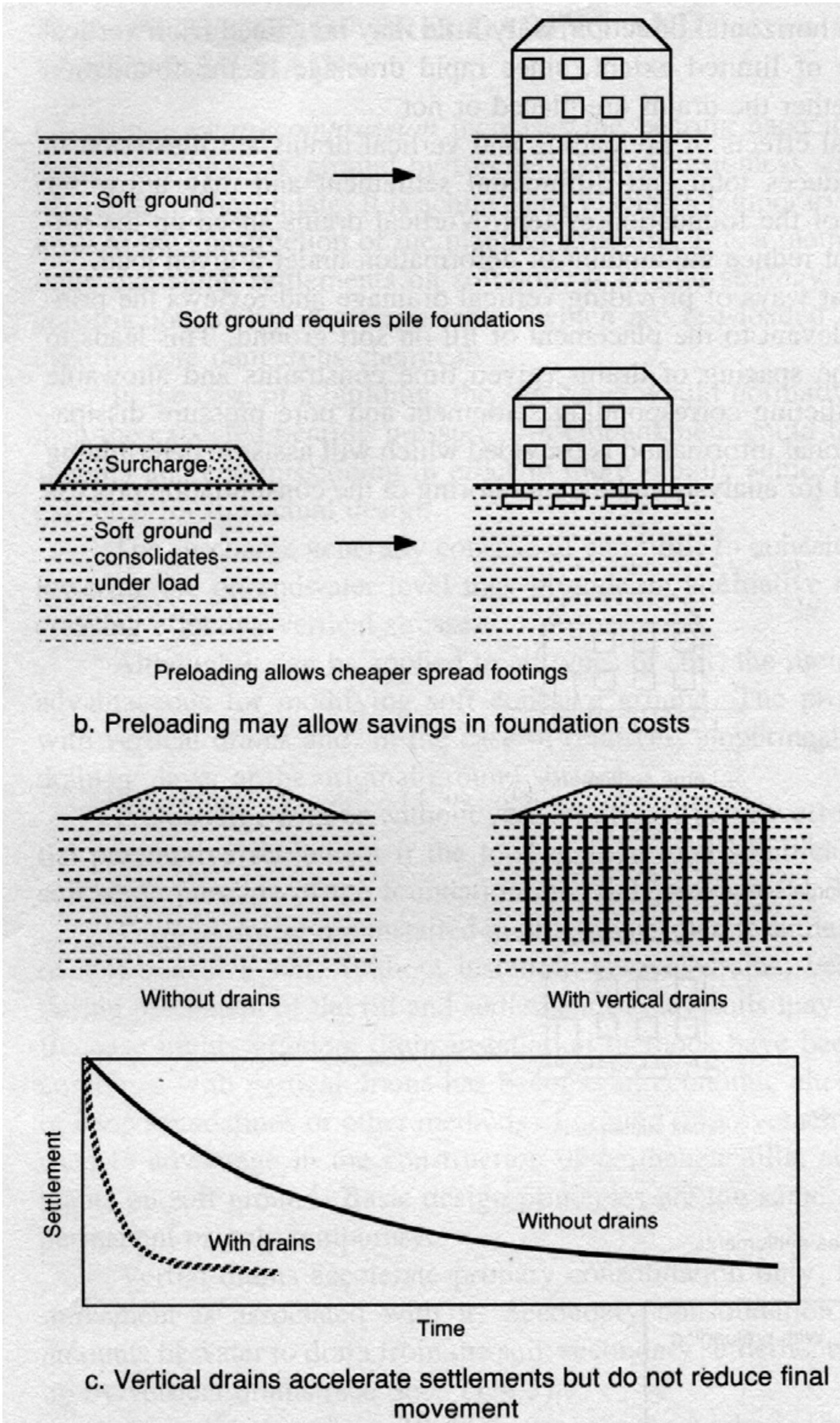


Figure 4.6 Preloading with Wick Drains (from Hausmann, 1990)

Preloading with Radial Drainage – Sand Drains

The primary difference from traditional consolidation is the time-rate. Since the drainage path is now both radial and maybe also vertical, the solution for time-rate must be amended. The drains are placed in either a square or triangular pattern, and this has an effect on the calculation. Figure 4.7 illustrated the patterns.

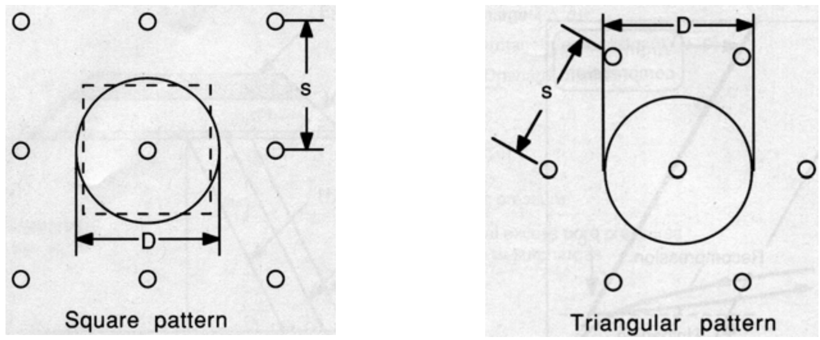


Figure 4.7 Vertical Drain Patterns

Therefore, the solution of radial drainage is;

$$U_r = 1 - e^{\left(\frac{-8T_r}{\alpha}\right)} \quad \text{where} \quad T_r = \frac{c_h t}{D^2}$$

U_r average degree of radial consolidation

T_r time factor

c_h coefficient of horizontal consolidation

D, d_e equivalent diameter of soil around drain

1.06S for triangular pattern, 1.13S for square pattern

n D/d d is drain diameter or equivalent diameter of wick drain

α $$n^2 \frac{\ln n}{\left(n^2 - 1\right)} - \frac{3n^2 - 1}{4n^2}$$

Alternatively:

$$U_r = 1 - e^{\left(\frac{-8T_r}{m}\right)}$$

$$m = n^2 \frac{\ln(n)}{(n^2-1)} - \frac{3n^2-1}{4n^2} \quad \text{where } n = d_e/2r_w$$

Combined vertical and radial drainage

Since, more than likely, vertical and radial drainage will occur simultaneously, the combined effects can be quantified as follows;

$$1 - U_{vr} = (1 - U_v)(1 - U_r)$$

Procedure for finding spacing:

1) Calculate Uv as from $T_v = \frac{c_v t}{H^2}$
2) Set U_{vr} = 0.9 (must be > U_r)
3) Rewrite

$$D^2\alpha = \frac{-8c_h t}{\ln(1 - U_r)} \quad \text{then solve for D}$$

4) Check S

Prefabricated Drains

As mentioned previously, wick drains are more commonly used for vertical drains. A wick drain consists of a plastic channel strip covered by a geotextile filter. Figure 4.8 and 4.9 show typical wick drain sections.

Equivalent Drain (Wick Drains)

In order to use the governing equations for radial drainage, some equivalencies must be established between rectangular wick drains and circular sand drains:

Equal void area $d_w = \frac{\sqrt{\frac{4Btn_d}{\pi}}}{n_s}$

B,t width, thickness of strip

n_d void area/total cross-section of strip

n_s sand drain porosity

Equal circumference $d_w = \frac{2(B+t)}{\pi} \therefore r_w = \frac{d_w}{2}$

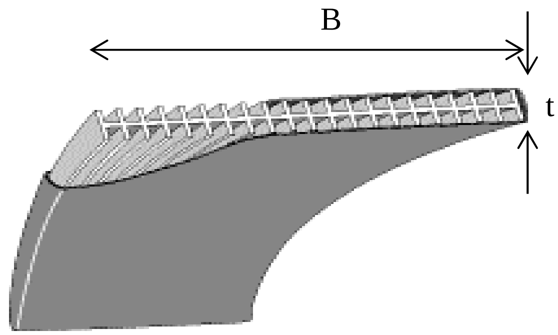


Figure 4.8 Prefabricated Vertical Drain

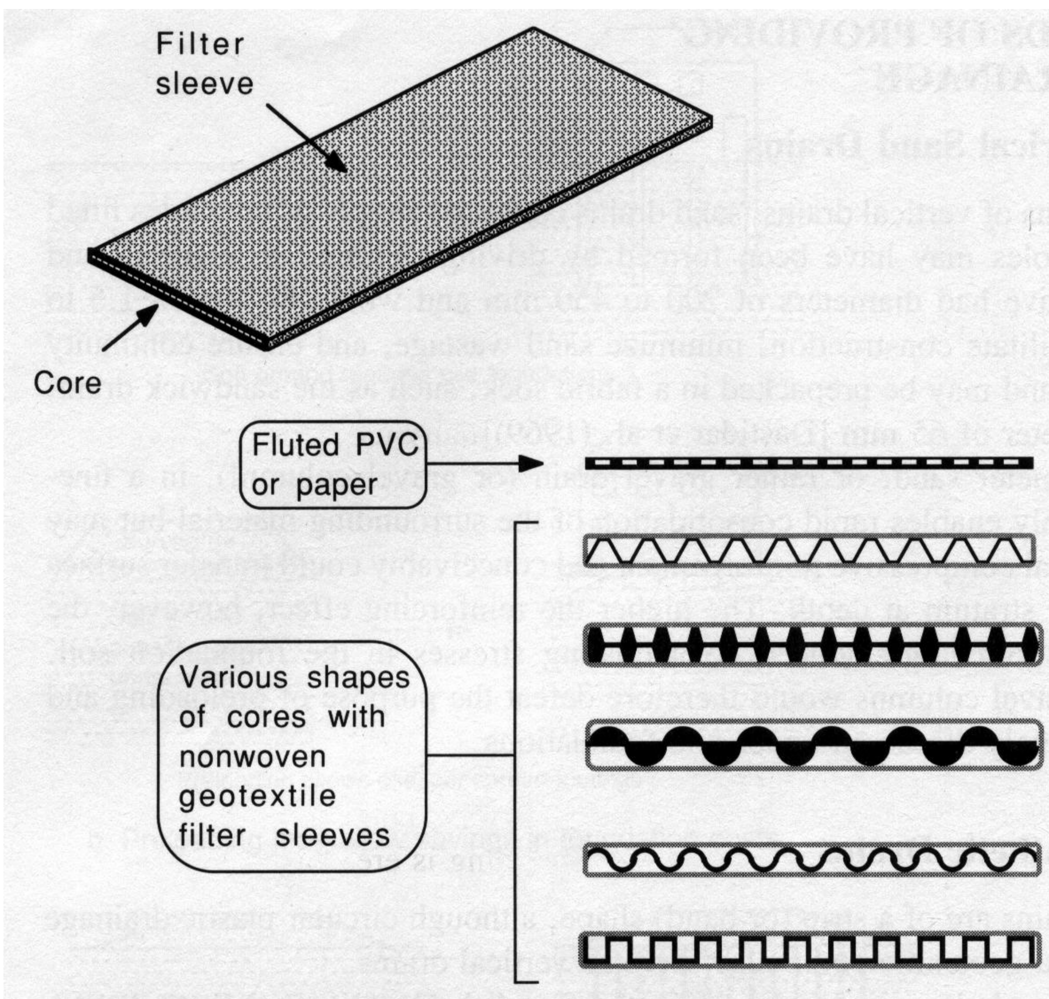


Figure 4.9 Types of Prefabricated Vertical Drains

EXAMPLE OF COMBINED RADIAL AND VERTICAL FLOW

Add sand drains to the previous problem r_w = 0.1m, D = 3m, $c_v = c_{vr} = c_h$,

As before, calculate required surcharge for 9 months.

1) Calculate $U_r = 1 - e^{\left(\frac{-8T_r}{m}\right)}$

a) $m = n^2 \frac{\ln(n)}{(n^2 - 1)} - \frac{3n^2 - 1}{4n^2}$ where n = $d_e/2r_w$ = 3/2(.1) = 15

$$m = 15^2 \frac{\ln(15)}{(15^2 - 1)} - \frac{3(15)^2 - 1}{4(15)^2} = 1.971$$

b) $U_r = 1 - e^{\left(\frac{-8T_r}{m}\right)}$ $T_r = \frac{c_h t}{D^2} = \frac{0.36(9)}{3^2} = 0.36$

$$U_r = 1 - e^{\left(\frac{-8(0.36)}{1.971}\right)} = 0.768 = 76.8\%$$

2) Combined $1 - U_{vr} = (1 - U_v)(1 - U_r)$ or $U_{vr} = 1 - (1 - U_v)(1 - U_r)$

$$U_{vr} = 1 - (1 - 0.6665)(1 - 0.768) = 92.3\%$$

3) But $U = \frac{\log \frac{P_o' + \Delta P_F}{P_0'}}{\log \frac{P_0' + \Delta P_F + \Delta P_S}{P_0'}} = \frac{\log \frac{210 + 115}{210}}{\log \frac{210 + 115 + \Delta P_S}{210}} = \frac{0.1897}{\log \frac{210 + 115 + \Delta P_S}{210}}$

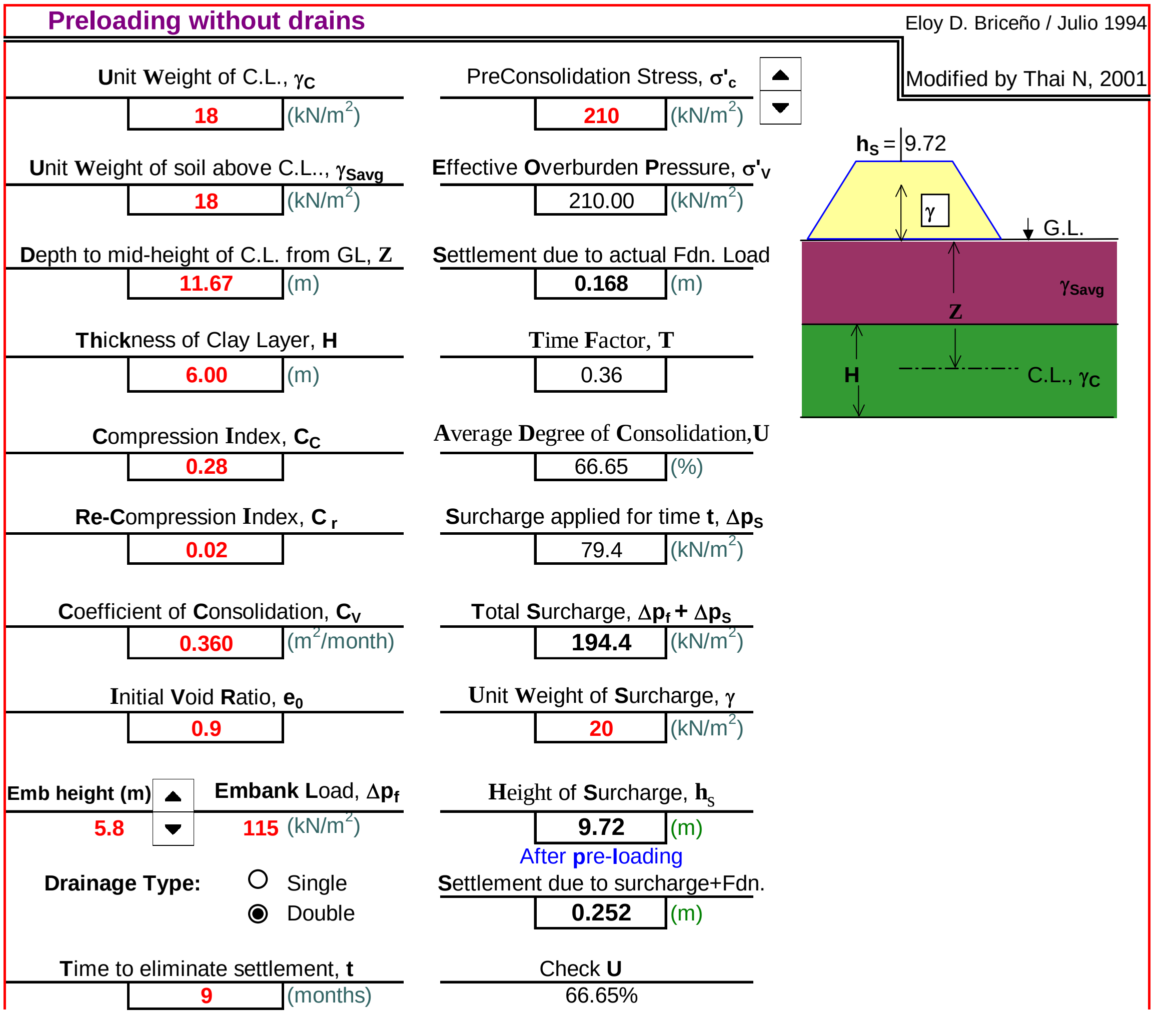
$$\log\left[\frac{(325 + \Delta P_S)}{210}\right] = \frac{0.1897}{0.923} = 0.2056$$

$$\frac{(325 + \Delta P_S)}{210} = 10^{0.2056} 1.6055$$

$$\Delta P_S = 12.15 \text{kN/m}^2$$

Thus, the surcharge required is 115 + 12.15 = 127.1 kN/m^2

Figure 4.10 shows the surcharge spreadsheet with the examples from previous sections.



Preloading without drains

Eloy D. Briceño / Julio 1994

Modified by Thai N, 2001

Unit Weight of C.L., γ_C: 18 (kN/m^2)

Unit Weight of soil above C.L.., γ_{Savg}: 18 (kN/m^2)

Depth to mid-height of C.L. from GL, Z: 11.67 (m)

Thickness of Clay Layer, H: 6.00 (m)

Compression Index, C_C: 0.28

Re-Compression Index, C_r: 0.02

Coefficient of Consolidation, C_V: 0.360 (m^2/month)

Initial Void Ratio, e_0: 0.9

Emb height (m): 5.8

Embank Load, Δp_f: 115 (kN/m^2)

Drainage Type: Single / Double (selected)

Time to eliminate settlement, t: 9 (months)

PreConsolidation Stress, σ'_c: 210 (kN/m^2)

Effective Overburden Pressure, σ'_V: 210.00 (kN/m^2)

Settlement due to actual Fdn. Load: 0.168 (m)

Time Factor, T: 0.36

Average Degree of Consolidation, U: 66.65 (%)

Surcharge applied for time t, Δp_S: 79.4 (kN/m^2)

Total Surcharge, $\Delta p_f + \Delta p_S$: 194.4 (kN/m^2)

Unit Weight of Surcharge, γ: 20 (kN/m^2)

Height of Surcharge, h_s: 9.72 (m)

After pre-loading

Settlement due to surcharge+Fdn.: 0.252 (m)

Check U: 66.65%

Figure 4.10 Surcharge XLS Spreadsheet

Preloading With Radial Drainage : Sand Drains

Continuing the previous problem……

Spacing between drains, **s**	2.65	(m)	**G**rid of **D**rains: △ =1, □ =2	2	
Dia.of effective radial drainage zone,d_e	2.99	(m)	**m**	1.969	
Radius of **S**and drains,r_w	0.10	(m)	**A**verage **D**egree of **C**onsolidation, U_r	76.95	(%)
n = (d_e/2r_w)	14.97		**A**verage **D**egree of **C**onsolidation, $U_{V,r}$	92.32	(%)
Cofficient of **C**onsolidation, C_V	0.360	(m^2/month)	**S**urcharge 2 b applied for time **t**, Δp_S	12.03	(kN/m^2)
Cofficient of **C**onsolidation, C_{Vr}	0.36	(m^2/month)	**T**otal **S**urcharge, $\Delta p_f + \Delta p_S$	127.03	(kN/m^2)
Time to eliminate settlement, **t**	9	(months)	**H**eight of **S**urcharge, h_S	6.35	(m)
Non-dimensional **T**ime factor,T_r	0.361				

subscript	Denotes
r	Radial drainage only
v	Vertical Drainage only

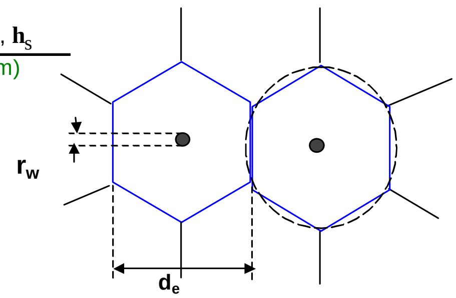


Figure 4.10 (continued)

Preloading With Radial Drainage : Wick Drains

Continuing the previous problem……

Parameter	Value	Unit
Spacing between drains, s	1.52	
Dia.of effective radial drainage zone, d_e	1.61	(m)
Width of Strip, B	0.095	(m)
Thickness of Strip, t	0.000	(m)
Equivalent Dia. Of drains, d_w	0.060	(m)
n = (d_e/2r_w)	26.641	
Cofficient of Consolidation, C_V	0.360	(m^2/month)
Cofficient of Consolidation, C_{Vr}	0.360	(m^2/month)
Time to eliminate settlement, t	9	(months)
Grid of drains: △=1, □ =2	1	
Vertical Time Factor, T	0.36	
Average Degree of Consolidation, Uv	66.65	(%)
Non-dimensional Time factor, T_r	1.248	
m (or α)	2.537	
Average Degree of Consolidation, U_r	98.05	(%)
Average Degree of Consolidation, $U_{V,r}$	99.35	(%)
Surcharge 2 b applied for time t, Δp_S	0.9	(kN/m^2)
Total Surcharge, $\Delta p_f + \Delta p_S$	115.93	(kN/m^2)
Height of Surcharge, h_S	5.80	(m)
Settlement due to Sur.+Fdn.	0.169	(m)
Check U	99.35%	

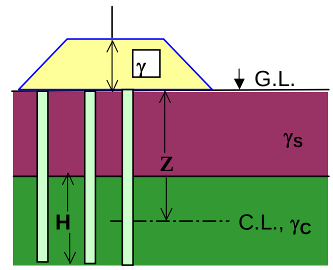


Figure 4.10 (continued)

Wick Drain Materials and Installation

Wick drains are installed using a mandrel rig. The come prefabricated on large rolls like spools of paper or fabric. Installation is rapid in weak soils and wick drains are easy to splice when necessary. Often times, a set of surface drains are employed to carry the extracted water when a preload embankment fill is placed. Figures 4.11 through 4.14 show the wick material, drain installation and the use of surface drains.

PERTINENT WEB LINKS FOR REVIEW

http://uswickdrain.wilmington.net/

http://www.nilex.com/

http://www.terrasystems-inc.com/wick.htm

http://www.americandrainagesystems.com/

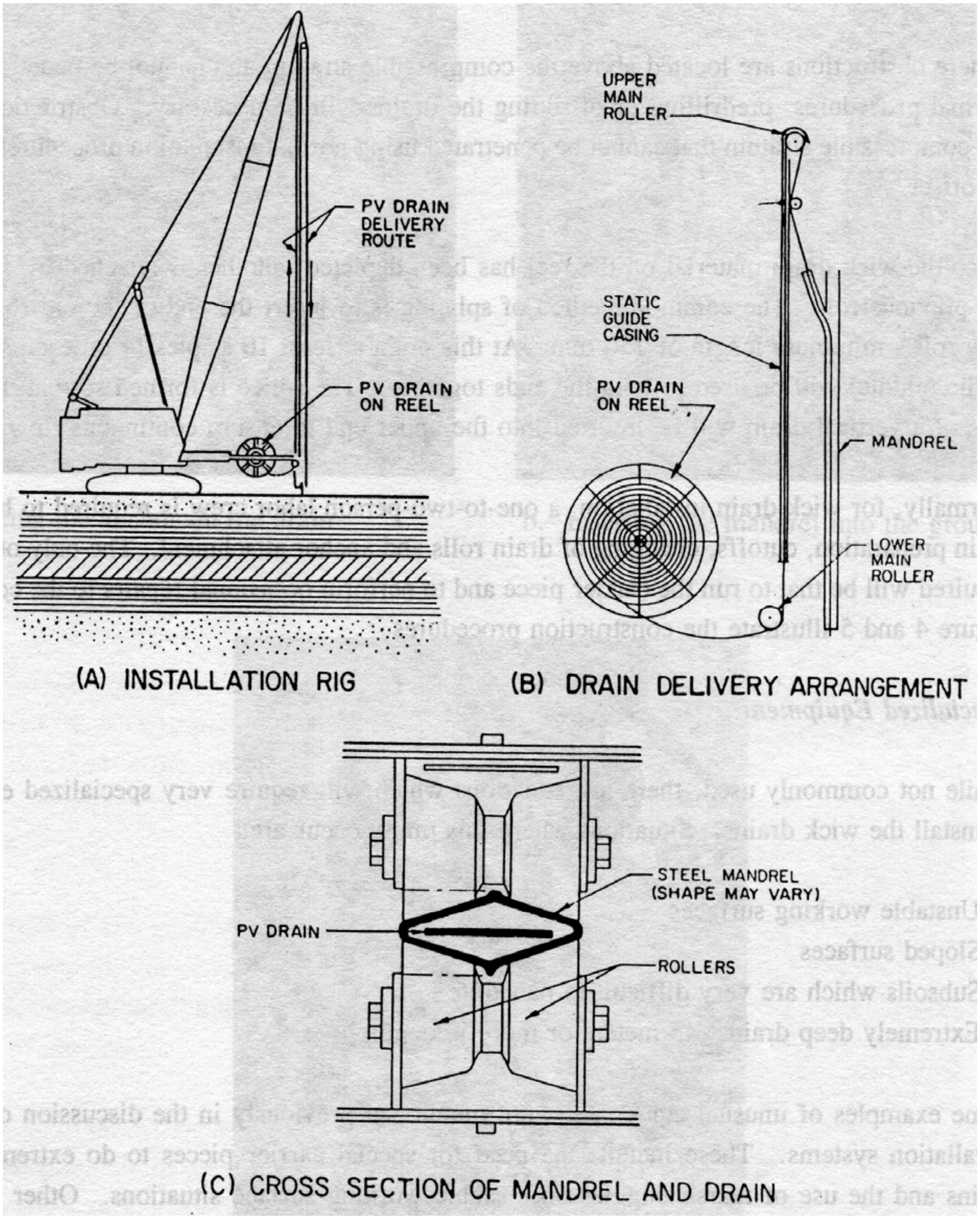
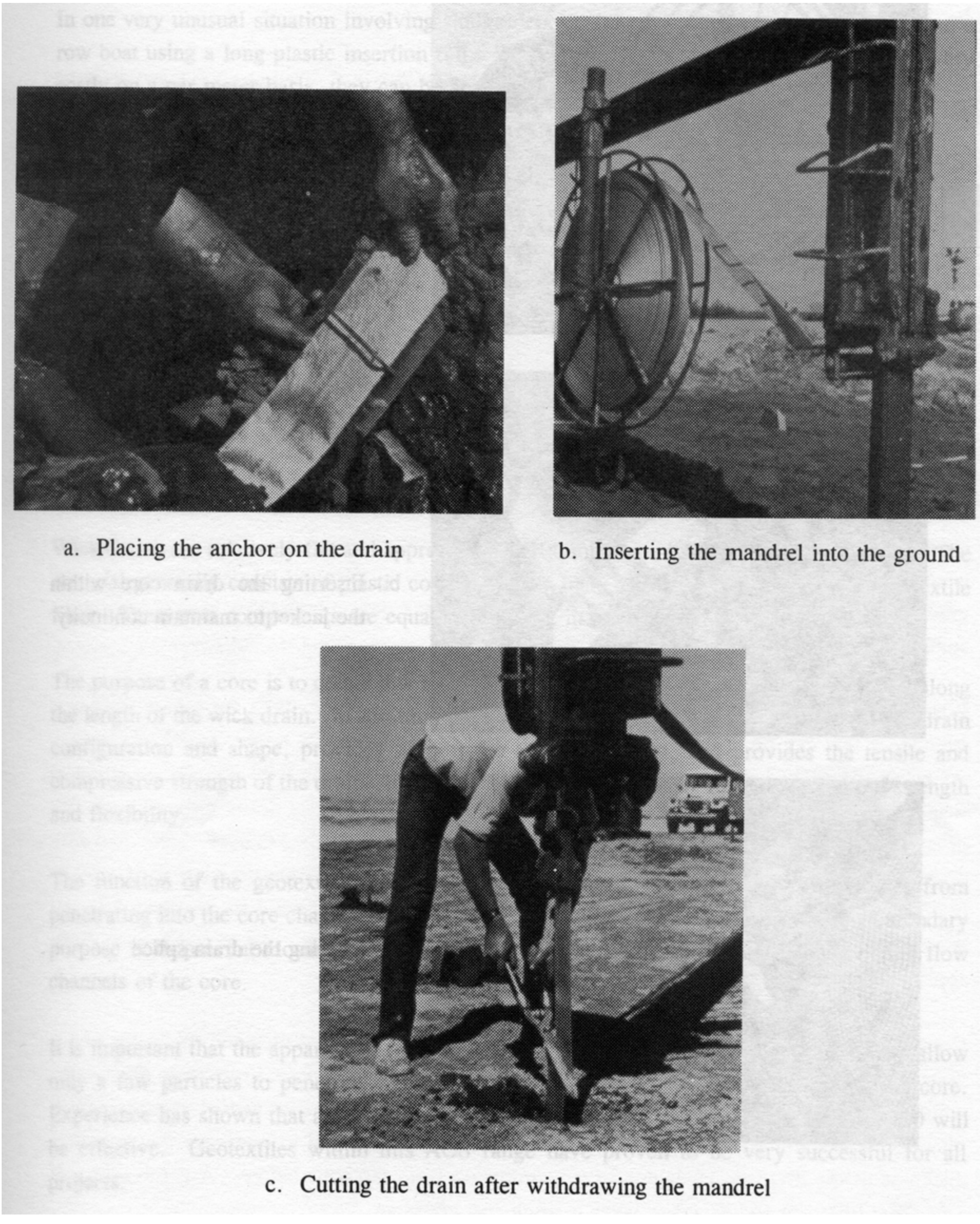


Figure 4.11 Installation of Wick Drains (from FHWA ,1999)



a. Placing the anchor on the drain

b. Inserting the mandrel into the ground

c. Cutting the drain after withdrawing the mandrel

Figure 4.12 Photographs of Wick Drain Installation (from FHWA,1999)

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 4.13 Photographs of Wick Drain Material and Splicing (from FHWA, 1999)

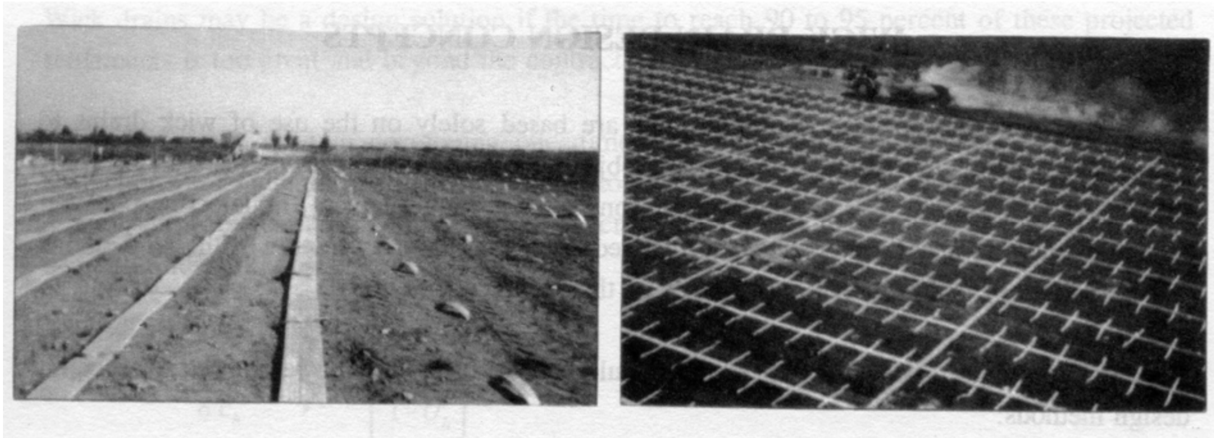


Figure 4.14 Horizontal Strip Drains to Collect Drainage (from FHWA, 1999)

REFERENCES

FHWA (1999). Ground Improvement Technical Summaries Demonstration Project 116, Vol. 1 and 2, Pub. No. FHWA-SA-98-086, Authors: Elias, V., Welsh, J., Warren, J. and Lukas, R.

Hausmann, M.R. (1990) Engineering Principles of Ground Modification, McGraw-Hill Publishing Co.

Rowe, P.W. (1968) "The Influence of Geological Features of Clay Deposits on the Design and Performance of Sand Drains," *Institution of Civil Engineers*, Proc. March. pp. 465-466.

Chapter 5

STONE COLUMNS

DESCRIPTION AND HISTORY

Since the early 1970's, stone column technology has become established in the United States as a viable ground improvement tool and has been applied extensively for remediation and new construction. Stone columns have a proven record of experience and are ideally suited for improving clays, silts, and loose silty sands (FHWA, 1999).

IN-SITU GROUND REINFORCEMENT

The concept involves replacement of 10 to 35 percent of the weak soil with stone—or sometimes with sand—in the form of columns. A hole is created in the ground by jetting, or other methods, then backfilled with stone compacted by impact and vibration. The soil is, thus, transformed into a stiffer composite mass of granular cylinders with intervening native soil providing lower overall compressibility and higher shear strength than those of the native soil alone (ASCE, 1987).

To date, stone columns have been used mainly to improve the bearing capacity and reduce the settlement of foundations and embankments, or to improve the stability of embankments and slopes. Other reported uses include liquefaction control (Engelhardt and Golding, 1975).

HISTORY

The densification of cohesionless granular soils with vibratory equipment is a well known construction procedure. The development of a special probe, called a Vibraflot was developed and patented in Germany in the early 1930's. Consequently non-cohesive soils could be densified to depths up to 10 meters. However, vibratory densification of <u>cohesive</u> soils is not viable by this method, due to their cohesive nature. Consequently, a variant of vibraflotation was developed in Germany in the 1950's to strengthen cohesive soils. This construction technique is termed stone columns.

It is interesting to note that the first documented use of stone columns was for the Taj Mahal in India, completed in 1653. This historic structure has been successfully supported for over three centuries by hand dug pits backfilled with stone. The concept of stone columns was also used in France in the 1830s to improve native soil (Barksdale and Bachus, 1983). Modern techniques were first implemented during the 1960s in Europe. After extensive use in Europe, the stone column technique was introduced into the United States in the 1970s but saw limited use in its first 12 years, with only 21 completed projects. However, by 1994, this number had increased over 20 times, due to the better understanding of the design concepts and economics of stone column techniques and the fact that more projects are being built on sites with poor soil (FHWA, 1999). Stone columns are constructed mainly by either the wet method (vibro-replacement) or the dry method (vibro-displacement). Rammed stone columns and dynamic replacement methods are also used but to a lesser extent (ASCE, 1987).

TYPES OF STONE COLUMN CONSTRUCTION

Vibro-Replacement (Top and Bottom Feed)

In the vibro-replacement (wet) process, a hole is jetted in the ground to the required depth using a vibratory probe (vibroflot). Upon reaching the desired depth, the uncased hole is flushed out and stone is added in increments compacted by vibration and by raising and lowering the jet probe (Figures 5.1 and 5.2). This action of the vibroflot tends to ram the stone into the sides of the hole. If the shear stresses generated by the process are so severe that the original soil collapses into the hole, the continuing water upflow removes the collapsed material to the surface allowing the stone to expand further until an equilibrium is reached as the stone is added and compacted in layers. The diameter of the column varies with depth. It is generally larger at the column base, at the ground surface and at softer soil strata (Munfakh, et al. 1984).

The vibro-replacement method is best suited for sites with very soft to firm soils (Cu = 15 to 50 kN/m^2) (300-1000 psf) and high groundwater table. In soft soils, the jet probe is left in the hole at all times and the flowing water is used for stabilizing the sides of the hole and washing out the fines to create a clean column as stone is tipped down the hole around the probe. The principal disadvantage of this technique is its impact on the environment. Since large quantities of water are used in the process, disposal of the excess water should follow acceptable environmental standards. In addition, pools of standing water around the constructed columns result in unsightly conditions of construction sites.

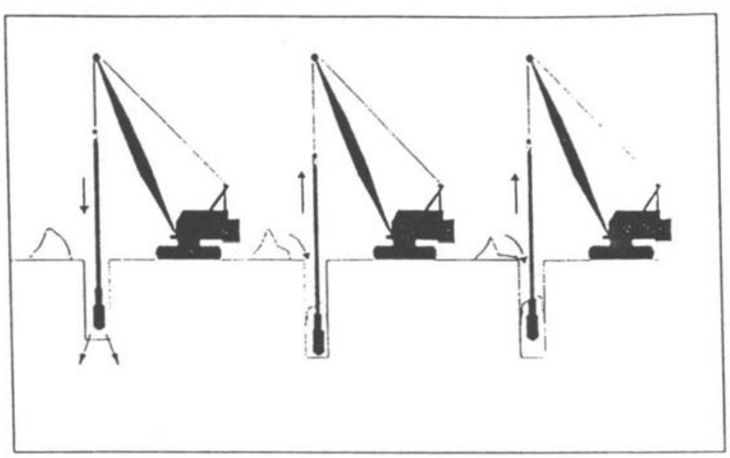


Figure 5.1 Top Feed Vibro-Replacement
(from: FHWA, 1999)

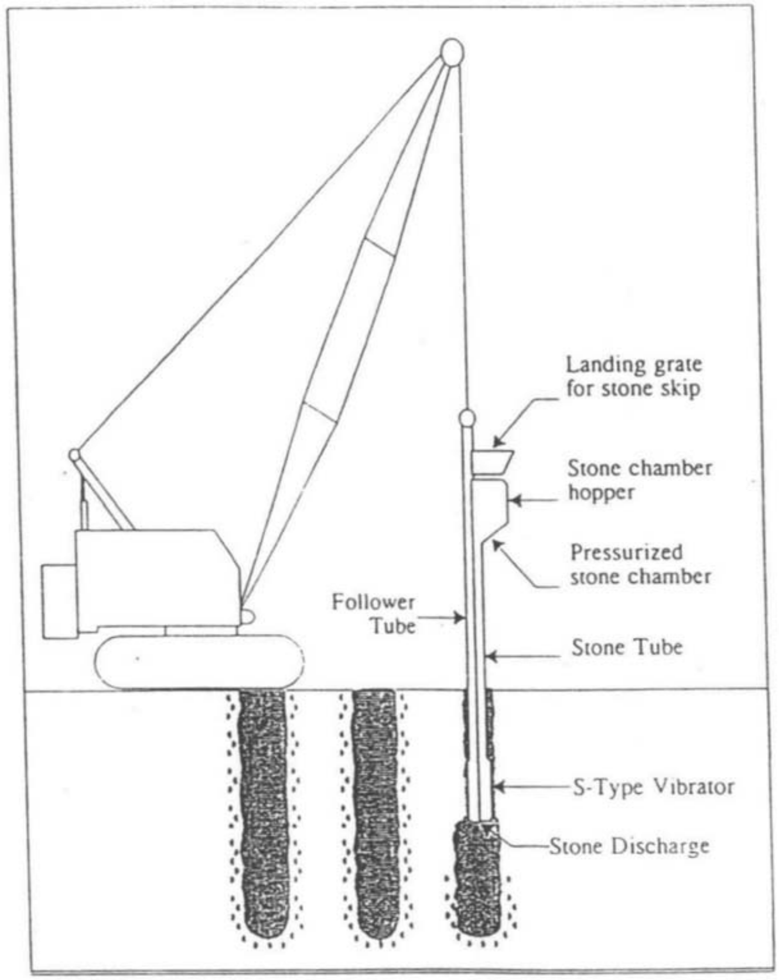


Figure 5.2 Bottom Feed Vibro-Displacement
(from: FHWA, 1999)

Vibro-Displacement

In the vibro-displacement (dry) method the probe displaces the soil laterally as it is advanced in the ground. Compressed air is usually used through the tip of the probe to facilitate penetration. Upon reaching the required depth, the probe is removed from the hole, backfill is placed in the hole in layers, and the probe is lowered again to displace the backfill laterally and downward creating a compacted stone column. The columns created by the dry process are usually smaller in diameter than those created by the wet process since they are usually used in stiffer soils and no in-situ material is removed from the hole in the process (ASCE, 1987).

Vibro-displacement is best suited for firmer soils (Cu = 30 to 60 kN/m^2) (600-1200 psf) with low sensitivity and low groundwater table. To avoid the need to remove the probe from the hole and to protect against collapse of the hole in softer soils, a machine was developed in Germany which allows the backfill to be discharged through the probe creating a compacted column while the probe remains in the ground (Figure 5.3). The use of the bottom-feed vibrator allows construction of stone columns by vibro-displacement in soft liquefiable soils. Jebe and Bartels (1983) describe in detail this "bottom-feed vibrator" and its modification for construction of both dry and mortared stone columns.

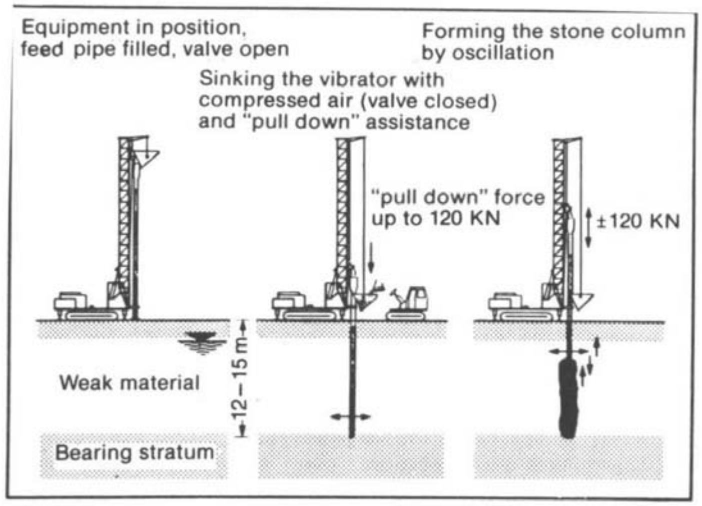


Figure 5.3 Bottom Feed Vibrator
(from: Jebe and Bartels, 1983)

Goughnour (1997) describes the RotoColumn™ method of installing stone columns. The method is the modification of the bottom-feed technique to accommodate stone columns in very soft cohesive soils. The heart of the system is a rotary installation impeller (Figure 5.4) located at the bottom of the feed pipe. Stone is fed down the feed pipe, and as the impeller rotates, stone is forced radially outward while more stone falls.

SOIL TYPE FEASIBILITY

Stone-column technology developed as a natural progression from vibro-compaction and extended vibro-system applications beyond the relatively narrow application of densification of clean, granular soils, as shown in Figure 5.5. The compactibility of a soil depends mainly on its grain size distribution. Soils with grain size distribution curves lying entirely on the coarse side of the hatched zone are generally readily compacted by the process known as vibro-compaction (vibraflotation). If the grain size distribution curve falls in the hatched zone, it is advisable to backfill with stone in lieu of sand during the compaction process to improve the contact between the vibrator and the treated soil. The many other soils with grain size distribution curves partly or entirely on the fine side of the hatched zone are not readily compactible by vibro-compaction. It is for these type of soils and their related problems that necessitated the development of stone column technology (FHWA, 1999).

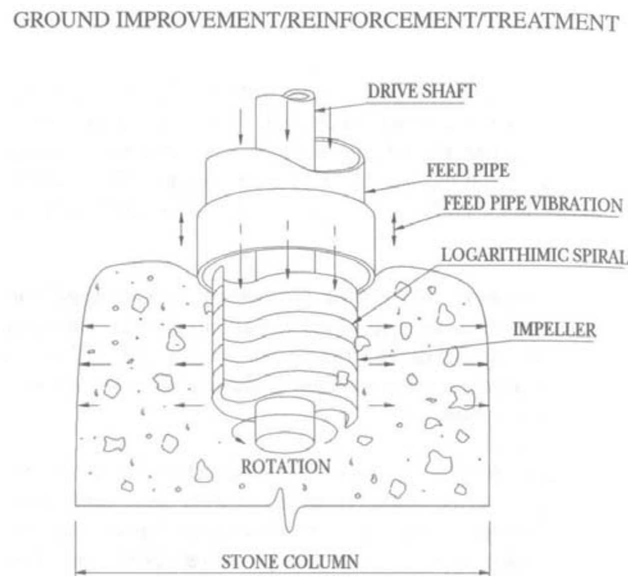


Figure 5.4 Rotary Installation Impeller
(from: Goughnour, 1997)

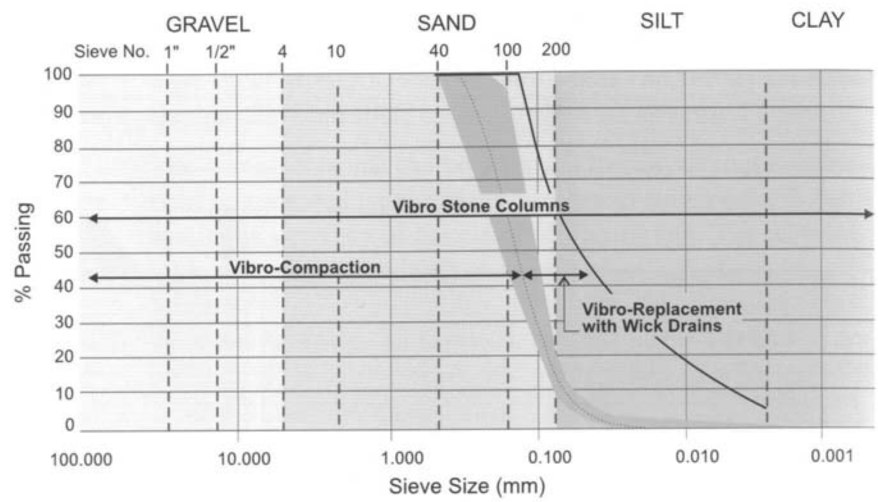


Figure 5.5 Grain Size Distribution Curves
(from: Hayward-Baker)

The most improvement is likely to be obtained in compressible silts and clays occurring near the surface and ranging in shear strength from 15 – 50 kPa (300 to 1000 psf). The greatest economic advantage is generally obtained if the depth to the bearing strata is between 20 to 30 ft (5-10 m). The lower shear strength limit is 8 kPa (150 psf) below which insufficient lateral support is provided and the stones merely displace into the soft surrounding soil. The upper shear strength is 50 – 100 kPa (1000-2000 psf, as above this strength the probe cannot penetrate (Barksdale and Bachus, 1983). Stone columns in peat are not recommended. Experience has shown that soils with less than 15% fines and a clay content less than 2% will densify by vibration. However, siltier and clayer soils do not react to vibration (Baez, 1995).

The effectiveness of stone columns for use in soft clays of high sensitivity is questionable due to the remolding effect of the installation process on the shear strength of the in-situ soil (Bauman and Bauer, 1974). In general, stone columns are not recommended for soils having sensitivities greater than 5 (ASCE, 1987).

FEASIBILITY EVALUATIONS

The stone column technique of ground treatment has proven successful in (1) improving stability of both embankments and natural slopes, (2) increasing bearing capacity, (3) reducing total and differential settlements, (4) reducing the liquefaction potential of cohesionless soils, and (5) increasing the time rate of settlement (FHWA, 1999).

The degree of densification resulting from the installation of vibro-systems is a function of soil type, silt and clay content, plasticity of the soil, pre-densification relative densities, vibrator type, stone shape and durability, stone column area, and spacing between stone columns. Experience has shown that soils with less than 15 percent passing a #200 sieve, and clay contents of less than 2 percent will densify due to vibrations. Clayey soils do not react favorably to the vibrations and the improvement in these soils is measured by the percent of soil replaced and displaced by the stone column, or pier.

A generalized summary of the factors affecting the feasibility of stabilizing soft ground with stone columns is as follows: (FHWA, 1999)

1. The allowable design loading of a stone column should be relatively uniform and limited to a maximum of 500 kN per column if sufficient lateral support by the in-situ soil can be developed.
2. The most significant improvement is likely to be obtained in compressible silts and clays occurring within 10 m of the surface and ranging in shear strength from 15 to 50 kN/m^2.
3. Special care must be taken when using stone columns in highly sensitive soils and in soils containing organics and peat lenses or layers with undrained shear strength of less than 10 kN/m^2. Because of the high compressibility and low strength of these materials, little lateral support may be developed and large vertical deflections of the columns may result. When the thickness of the organic layer is greater than 1 to 2 stone column diameters, the ability to develop consistent column diameters becomes questionable.
4. Ground improved with stone columns reduces settlements typically from 30 to 50 percent of the unimproved ground response.
5. Stone columns have been used in clays having minimum (not average) undrained shear strengths as low as 7 kN/m^2. Due to the development of excessive resistance to penetration of the vibrator and economic considerations, a practical upper limit is in the range of an undrained strength of 50 to 100 kN/m^2. Clays with greater shear strengths may, in fact, be strong enough to withstand the loads without ground improvement. If stone columns are used in these stiff soils or through stiff lenses, the column hole is

commonly prebored, which is often the case in landslide projects. This situation will result in a significant additional cost.

6. Individual stone columns are typically designed for a bearing loads 20 to 30 tons per column. The ultimate capacity of a group of stone columns is predicted by estimating the ultimate capacity of a single column and multiplying that capacity by the number of columns in the group.
7. Stone columns have been used effectively to improve the stability of slopes and embankments. The design is usually based on conventional slip circle or wedge analyses utilizing composite shear strengths.

Improvement of Settlement Characteristics

Stone columns act similarly to prefabricated, vertical drains (wicks) in decreasing the distance which water has to flow in the radial direction for primary consolidation to occur. As a result, installation of stone columns can, in the absence of natural drainage layers within cohesive soils, significantly decrease the time required for primary consolidation. Under these conditions, the presence of stone columns will greatly accelerate the gain in shear strength of cohesive soils as primary consolidation occurs. In addition, total settlement is reduced as the stone columns carry a greater portion of the total surface load.

DESIGN OF STONE COLUMNS

Soil-Column Interaction (ASCE, 1987)

Stone columns are stiffer than the in-situ soil they replace. Because the column is cohesionless, its stiffness depends upon the lateral support given by the soil around it. If that support is inadequate, the column fails by bulging. The stability of a soil-column composite system also depends on whether shear action (skin friction) develops between the column and the soil surrounding it. For instance, if the footing load is applied to the column alone leaving the surrounding ground unloaded, or if the column is forced to settle at the top unevenly with the surrounding soil under the influence of a wide-loaded area, shear forces may be induced along the column at the column-soil interface. In such cases, a column may fail as a pile because of insufficient skin friction and end bearing. Stone columns should be analyzed for both modes of failure—bulging or shear (Greenwood and Kirsch, 1984).

Soft compressible soils undergo much lower settlements when they are stiffened by stone or sand columns. Bulging of the column under the load causes horizontal compression of the soil between columns which provides additional confinement for the stone. An equilibrium is eventually reached resulting in reduced vertical movement when compared to unreinforced soil.

Because the rigidity of the stone column is substantially higher than that of the surrounding soil, a larger portion of the applied load is transferred to the stone. A blanket of sand and gravel or a semi-rigid mat of reinforced earth is usually placed above the stone column-reinforced soil. This mat facilitates transfer of superimposed loads to the stone columns by arching over the in-situ soil. with time, as the surrounding clay consolidates, further load transfer takes place from the native soil to the stone columns by negative friction resulting in additional reduction in soil settlement (Munfakh et al. 1984).

Hughes, et al. (1976) defines a critical length for an isolated stone column considering the column as a pile. Beyond that length, the column does not contribute extra benefit in terms of bearing capacity but it continues to reduce settlement by penetrating to a firmer layer. With typical soil and column parameters, the critical depth is usually about four column diameters (Mattes and Poulos, 1969).

Fundamental Aspects of Design

Analysis of stone column-soil structure is subject to the following parameters: (1) column diameter, (2) spacing, (3) angle of internal friction of the stone, (4) shear strength of the surrounding soil, (5) stress ratio between column and soil, and (6) stress-strain relationship of both stone and surrounding soil (ASCE, 1987).

The column diameter is dictated by the desired level of improvement, the method of installation, the stone size and the strength of the in-situ soil. Reported column diameters range from 1.5 ft. (0.45 m) to 4.0 ft. (1.2 m). Besancon et al. (1984) developed a graphical correlation between the column diameter and the undrained shear strength of the soil using actual reported case applications (Figure 5.6). The lower portion of their proposed graphical band corresponds to stone columns constructed using less than 40-mm size material while the upper portion represents columns with materials up to 100-mm in size.

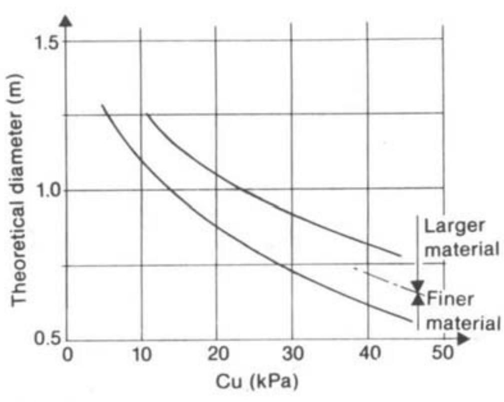


Figure 5.6 Effect of Soil Strength of the Theoretical Column Diameter

(Besancon et al., 1984)

Square or rectangular grid patterns with center-to-center column spacings of 5 to 12 ft (1.5 m to 3.5 m) are used. Columns may be used in clusters or rows to support spread footings or wall foundations or distributed over a wide area for support of rafts or embankments.. The column spacing is a function of the desired improvement, the construction process, and the sensitivity of the in-situ soil.

The angle of internal friction of the stone column depends on the size and shape of the stone, the installation process and the infiltration of the native soil between stone particles. An angle of internal friction of 35 degrees can be assumed for evaluation of stone column horizontal resistance. Higher values of 40-45 degrees have been used subsequently, based mainly on the results of direct shear tests performed in the field on constructed columns (Munfakh et al., 1983). Based on parametric studies of stone-column applications, Greenwood and Kirsch (1984) conclude that friction angle variation of about 5 degrees has comparatively little influence on the ultimate capacity and settlement of stone columns. The friction angle, however, may have a large effect on the horizontal shear resistance of the stone column reinforced soil. Analysis of the reinforcing effect of stone and sand columns is usually performed on the basis of the undrained shear strength of the clay.

PRELIMINARY DESIGN

Stone columns are typically selected to increase bearing capacity, reduce settlement, accelerate consolidation time rate, increase shear strength, reduce liquefaction potential or any combination of the above (FHWA, 1999).

Unit Cell Concept

For purposes of settlement and stability analyses it is convenient to associate the tributary area of soil surrounding each stone column with the column illustrated in Figures 5.7 and 5.8. Although the tributary area forms a regular hexagon about the stone column, it can be closely approximated as an equivalent circle having the same total area. The resulting equivalent cylinder of material having a diameter D_e enclosing the tributary soil and one stone column is known as the *unit cell*. The stone column is concentric to the exterior boundary of the unit cell (Barksdale and Bachus, 1983).

Area Replacement Ratio

The volume of soil replaced by stone columns has an important effect upon the performance of the improved ground. To quantify the amount of soil replacement the *Area Replacement Ratio*, a_s , is defined as the fraction of soil tributary to the stone column replaced by the stone:

$$a_S = \frac{A_S}{A} \qquad (1)$$

where A_S is the area of the stone column after compaction and A is the total area within the unit cell (Barksdale and Bachus, 1983). Typical ratios used are in the range of 0.10 to 0.40. The literature also describes the ratio a_S the *area improvement ratio* which is the inverse of an area replacement ratio.

Spacing and Diameter

Stone column diameters vary between 0.45 m and 1.2 m, but are typically in the range of 0.9 to 1.1 m for the dry method and somewhat larger for the wet method.

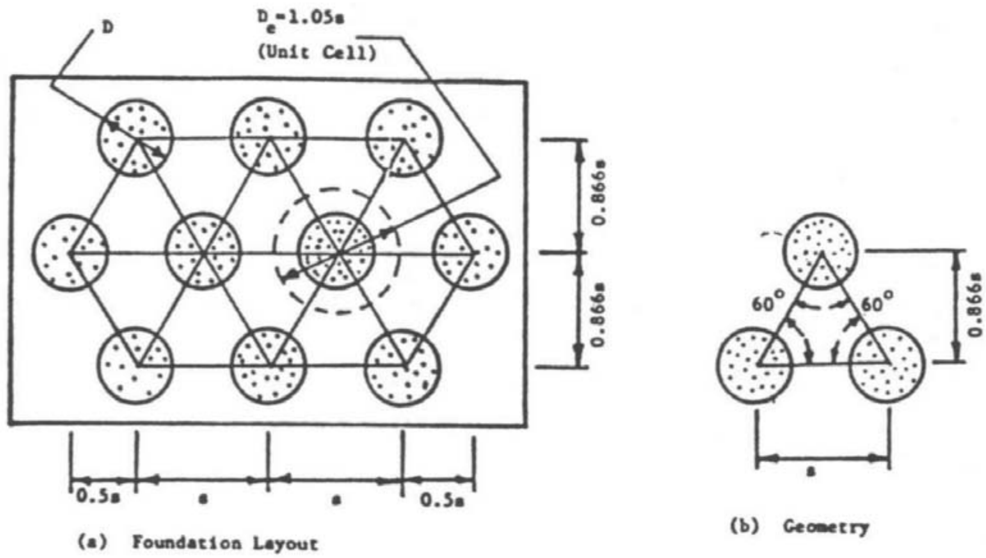


Figure 5.7 Equilateral Triangular Pattern of Stone Columns
(from: FHWA, 1999)

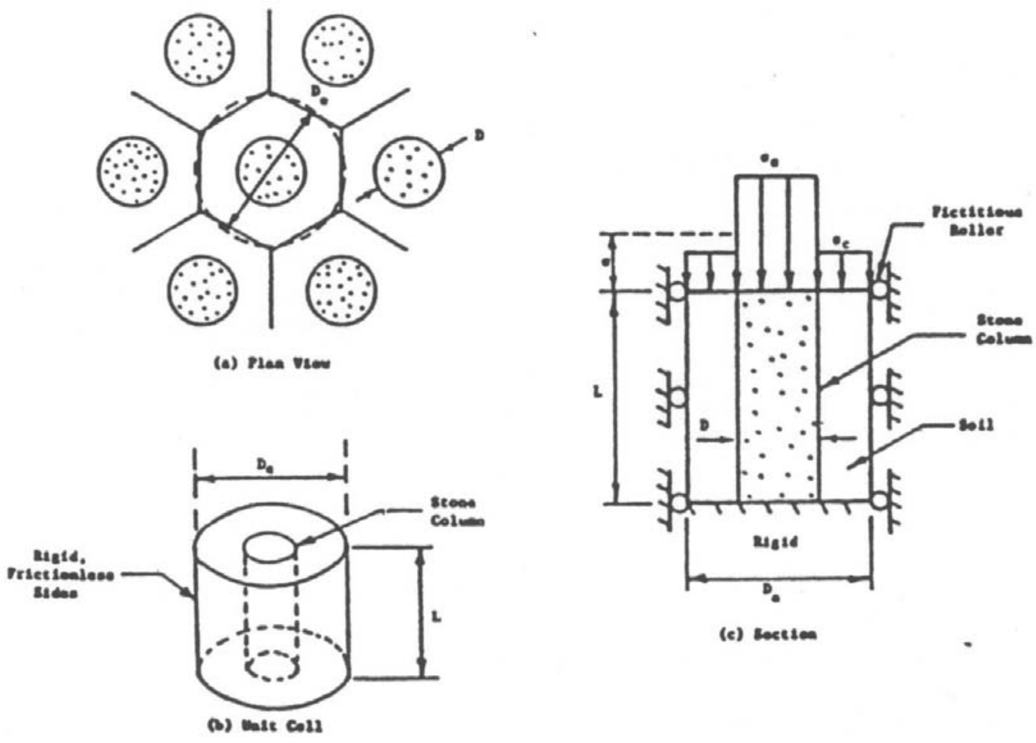
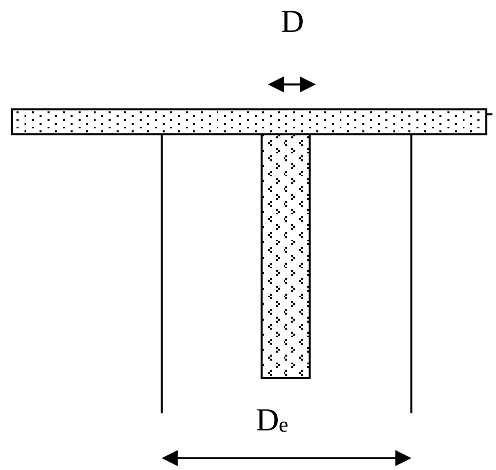


Figure 5.8 Unit Cell Idealization
(from: FHWA, 1999)

Triangular, square or rectangular grid patterns are used with center to center column spacing of 1.5 to 3.5 m. For footing support they are installed in rows or clusters. For both footing or wide area support they should extend *beyond the loaded area*.

For square grid patterns, the equivalent diameter, D_c, of the area tributary to the column with diameter D is; where S = spacing between columns.



$$D_e = 1.13\ S$$

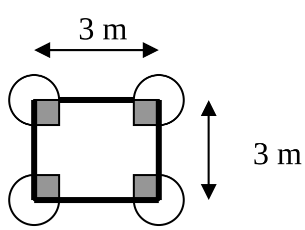
and for triangular patterns

$$D_e = 1.05\ S$$

Consequently, the area replacement ratio, a_S, is

$a_S = C(D/S)^2$ where $C = \pi/4$ for square patterns, and $C = \pi/2\sqrt{3}$ for triangular patterns.

Example: What is the area replacement ratio for a 1 m diameter stone column placed in a triangular grid pattern with a spacing of 10 m?



(1) $a_S = A_S/A$

(2) A_S = ¼ of a column × 4 columns =

$$\frac{\pi d^2}{4} = \frac{\pi (1)^2}{4} = 0.79$$

(3) $A = 3\text{ m} \times 3\text{ m} = 9\text{ m}^2$

(4) $a_S = 0.79/9 = 0.087$

or $a_S = C\,(D/S)^2 = \frac{\pi}{4}\,(1/3)^2 = 0.087$

Stress Ratio

The relative stiffness of the stone column to the in-situ soil as well as the diameter and spacing of the columns determines the sharing of the imposed area vertical load between the column and the in-situ soil (FHWA, 1999).

Since the deflection in the two materials is approximately the same, equilibrium considerations indicate the stress in the stiffer stone column must be greater than the stress in the surrounding soil. The assumption of equal deflection is frequently referred to as an equal strain assumption which both field measurements and finite element analyses have indicated to be valid.

The stress concentration or stress ratio n (stress in stone column divided by stress in in-situ soil) is dependent upon a number of variables including the relative stiffness between the two materials, length of the stone column, area ratio and the characteristics of the granular blanket placed over the stone column. Measured values of stress ratio have generally been between 2.0 and 5.0, and theory indicates this concentration factor should increase with time. Since secondary settlement in reinforced cohesive soils is greater than in the stone column, the long-term stress ratio in the stone column should be larger than at the end of primary settlement.

For preliminary design, the determination of a design stress ratio is the key element in stone column design and unfortunately, it is based largely on experience, although theoretical solutions are available (Barksdale and Bachus, 1983; Priebe, 1995).

A high stress ratio (3 to 4) may be warranted if the in-situ soil is very weak and the spacing very tight. For stronger in-situ soils and large spacings lower bound stress ratios (2 to 2.5) are indicated. For preliminary design a ratio of 2.5 is often conservatively used for stability and bearing capacity calculations.

Once a stress ratio has been assumed or determined, the stress on the stone column, σ_S, and on the surrounding soil, σ_C , can be calculated for each replacement ratio, a_S , and any average stress condition, q, that would exist over the unit cell as follows:

$$n = \frac{\sigma_S}{\sigma_C} \tag{2}$$

For equilibrium of vertical forces for a given a_S

$$q = \sigma_S (a_S) + \sigma_C (1 - a_S) \tag{3}$$

For a given stress concentration factor:

$$\sigma_C = \frac{q}{[1 + (n-1) a_S]} \tag{4}$$

$$\sigma_S = \frac{n q}{[1 + (n-1) a_S]} \tag{5}$$

Bearing Capacity

In determining the ultimate bearing capacity of a stone column or a stone column group, the possible modes of failure to be considered are illustrated in Figures 5.9, 5.10, and 5.11. Caution should be given to avoiding local bulging failures due to very weak, or organic layers of limited thickness (Figure 5.10). Bulging would have an effect on the time rate and magnitude of settlement and may be of concern with respect to stability and stone column shear strength. Use of a bulging analysis for a single column to predict group behavior gives an approximate conservative solution (FHWA, 1999).

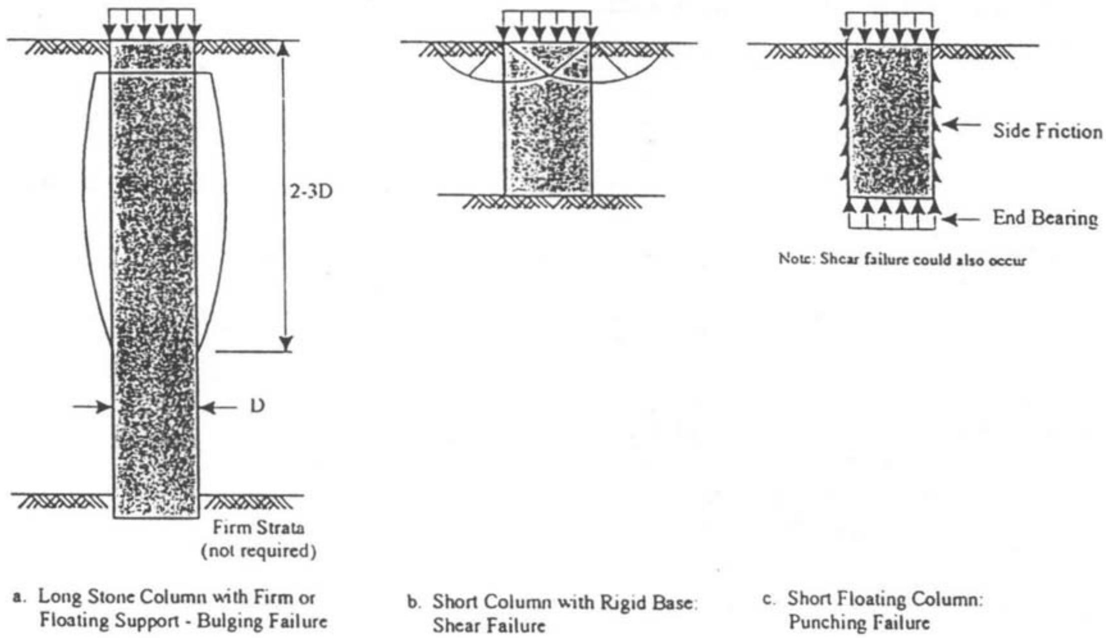


Figure 5.9 Failure Modes of a Single Stone Column in a Homogenous Soft Layer

(from: FHWA, 1999)

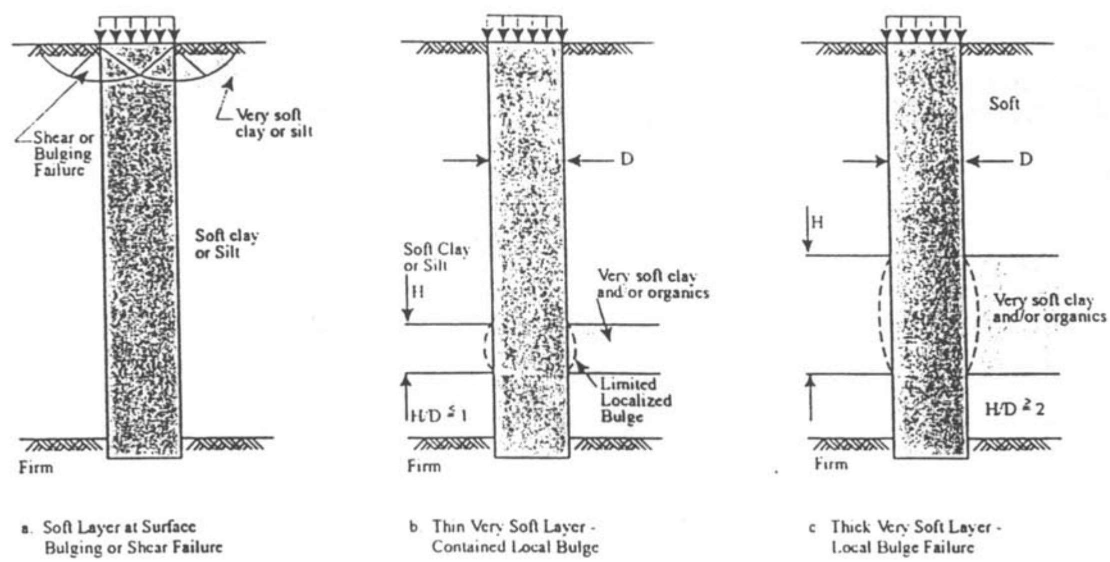


Figure 5.10 Failure Modes of a Single Stone Column in a Nonhomogenous Cohesive Soil

(from: FHWA, 1999)

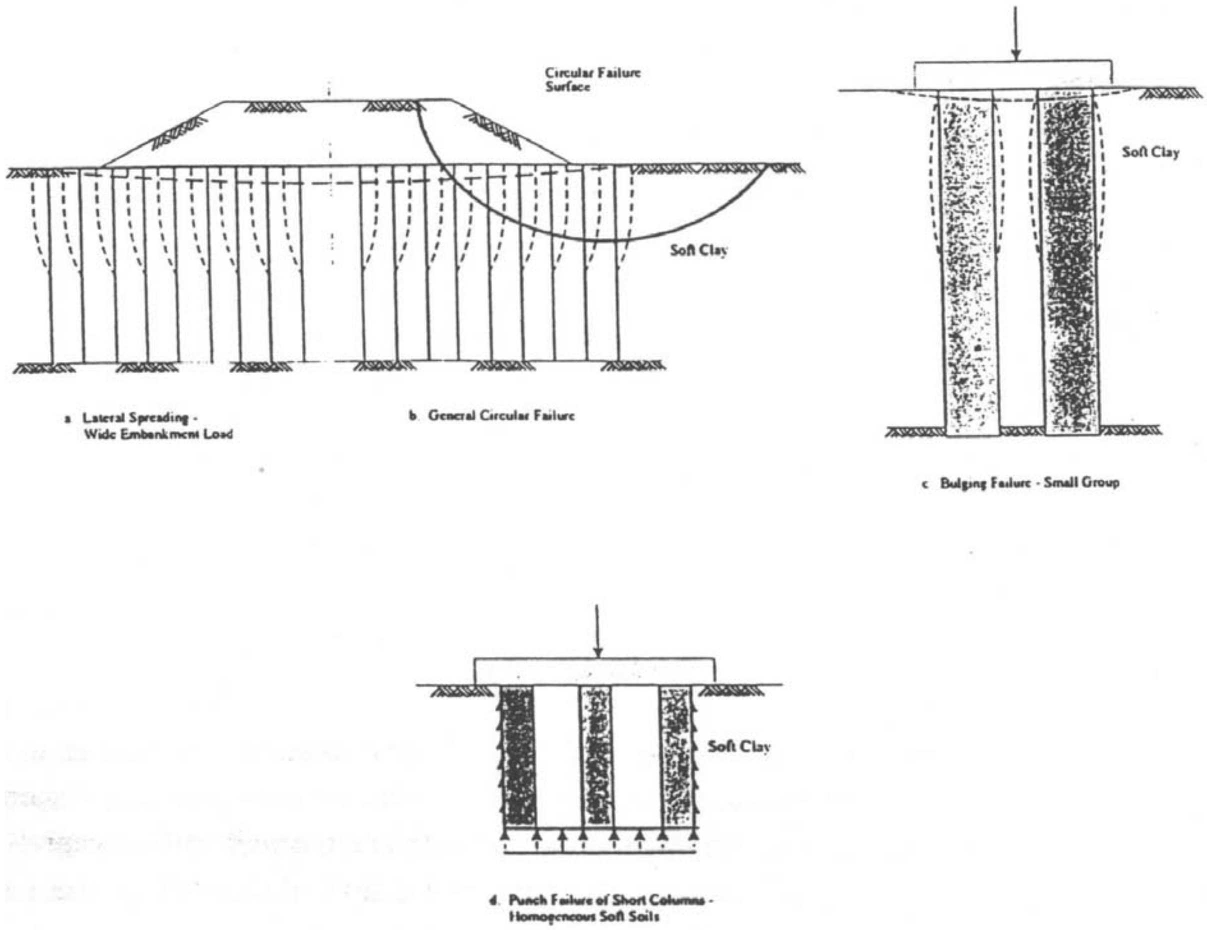


Figure 5.11 Failure Modes of Stone Column Groups
(from: FHWA, 1999)

The rational prediction of the bearing capacity of stone column groups loaded by either a rigid foundation or a flexible load, as an embankment, is still in the development stage. As a result, past experience and engineering judgement should be used in addition to theory when selecting a design stone column load.

Frequently, the ultimate capacity of a stone column group is predicted by multiplying the single column capacity by the number of columns in the group. small scale model studies using a rigid footing indicate this approach is probably slightly conservative for soft cohesive soils. The bearing capacity of an isolated stone column or a stone column located within a group can be expressed in terms of an ultimate stress applied over the stone column.

Typical single column design loads of 20 to 30 tons (200 - 300 kN) can be used in soft to medium stiff clays.

Method Using Bearing Capacity Factor

The load carrying capacity of an isolated column can be calculated as follows:

$$\sigma_v = \frac{\tilde{N}_{SC}\, C_U}{F.S.}$$

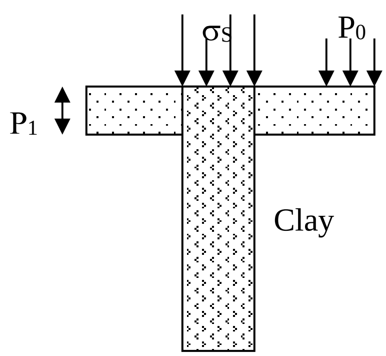
where $\tilde{N}_{SC}$ is a bearing capacity factor for stone columns and F.S. is the factor of safety. Mitchell (1981) recommends using $\tilde{N}_{SC}$ of 25 for vibro-replacement columns. Barksdale and Bachus (1983) propose a range between 18 to 22 depending upon the soil stiffness. Consequently, soils with organics or other soft clays, would be expected to have a smaller value of N_C compared to stiffer soils. For soils having a reasonably high initial stiffness, an N_C of 18 is recommended. Low stiffness soils would include peats, organic cohesive soils, and very soft clays with plasticity indices greater than 30. High stiffness soils would include inorganic soft-to-stiff clays and silts. The recommended values of $\tilde{N}_{SC}$ are based on a back-analysis of field test results. In this analysis, the strengths of both the soil and stone column were included. A factor of safety of 3 is recommended for design.

<u>Hayward-Baker Suggested Method (Braun, 1978 and Priebe 1976)</u>

Input:	Soil:	Su	=	undrained shear strength
	Stone:	d	=	diam. of column
		E_S	=	compression modulus (oedometer)
		ϕ_S	=	friction angle
		μ	=	Poissons ratio (≈ 1/3)
		a_1 b	=	spacing & fdn area
		γ	=	unit wt.

Explanation of the Single Calculations Steps

Bearing Capacity



1. Obtain the undrained shear strength C_u from the soil report.
2. Determine angle φ_S of backfill material from Table 5.1.
3. Assume the diameter of stone column d = 2-3.5 ft and calculate the stone column area A_S. Varies with soil strength. Weak soil – large d, stiff soil – small d.
4. Determine the effective overburden pressure on the cohesive soil level $q = P_1 \gamma + P_o$
5. Calculate the ratio q/C_u and using φ_S obtain the shear angle δ from Figure 5.12. If q = 0 use Figure 5.13.
6. Calculate σ_S / C_u from equation 1.
7. Calculate $P = \sigma_S A_S$, the initial bearing capacity.
8. Determine the design load P_D with a safety factor $\eta = 2$
9. Determine number of columns by dividing the foundation load by the design load P_D.

For stone columns not incorporating a sand leveling pad, Figure 5.13 is used.

According to Brauns (1978) 3-dimensional passive earth pressure theory for Stone Columns.

δ	=	Angle of shear failure from Figure 5.12 or 5.13
σ_S	=	Initial Ultimate Bearing Stress
q	=	Imposed Load
A_S	=	Stone Column Area
A_S	=	$\pi d^2/4$
φ_S	=	Angle of Friction in Stone Column
c_u	=	Undrained Shear Strength
δ_S	=	$45° + \varphi_S/2$

$$\frac{\sigma_S}{c_u} = \left(\frac{9}{c_u} + \frac{2}{\sin 2\delta}\right) g \left(1 + \frac{\tan \delta_S}{\tan \delta}\right) \tan^2 \delta_S$$

DESIGN LOAD $P_D = \frac{\sigma_S \cdot A_S}{\eta}$ SAFETY FACTOR $\eta = 2.0$

Note: Figure 5.12 is valid for $q > 0$; Figure 5.13 is valid for $q = 0$

Table 5.1 Average Design Parameters

Soil	Undrained Shear Strength c_u		Drained Shear Strength c_d		Elastic Modulus E_S	
	kN/m^2	psf	kN/m^2	psf	N/cm^2	t/ft^2
Clay Stiff	50-100	1000-2000	25	500	500-1000	50-100
Clay Firm	25-50	500-1000	10	200	250-500	25-50
Clay Soft	10-25	200-500	0	0	100-250	10-25
Sandy Clay Firm	50-100	1000-2000	5	100	500-2000	50-200
Sandy Clay Soft	10-25	200-500	0	0	400-800	40-80
Silt Stiff	30-50	600-1000	2	40	500-1000	50-100
Silt Soft	10-30	200-600	1	20	200-400	20-40
Organic Silt Soft	10-25	200-500	10	200	200-500	20-50
Organic Clay Soft	10-20	200-400	10	200	50-300	5-30
Peat	–	–	15	300	30-100	3-10

ANGLE OF FRICTION φ_S FOR BACKFILL MATERIAL

Sand Round	Sand Sharp	Gravel	Crushed Stone	Crushed Hard Stone
35°	38°	40°	42.5°	45°

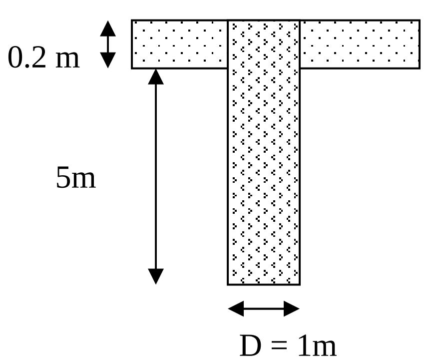
INITIAL BEARING CAPACITY

Design Load $P_D = \frac{\varphi_S \square A_S}{\eta}$

$\eta = 2$

$q = 0$

Example: Estimate the bearing capacity for a single stone column as shown



Unit weight of sand leveling pad $\gamma = 17$ kN/m^3

Clay: $C_u = 15$ kPa, $\gamma_{clay} = 14$ kN/m^3

Stone Column 1 m diameter, $\phi_S = 45°$

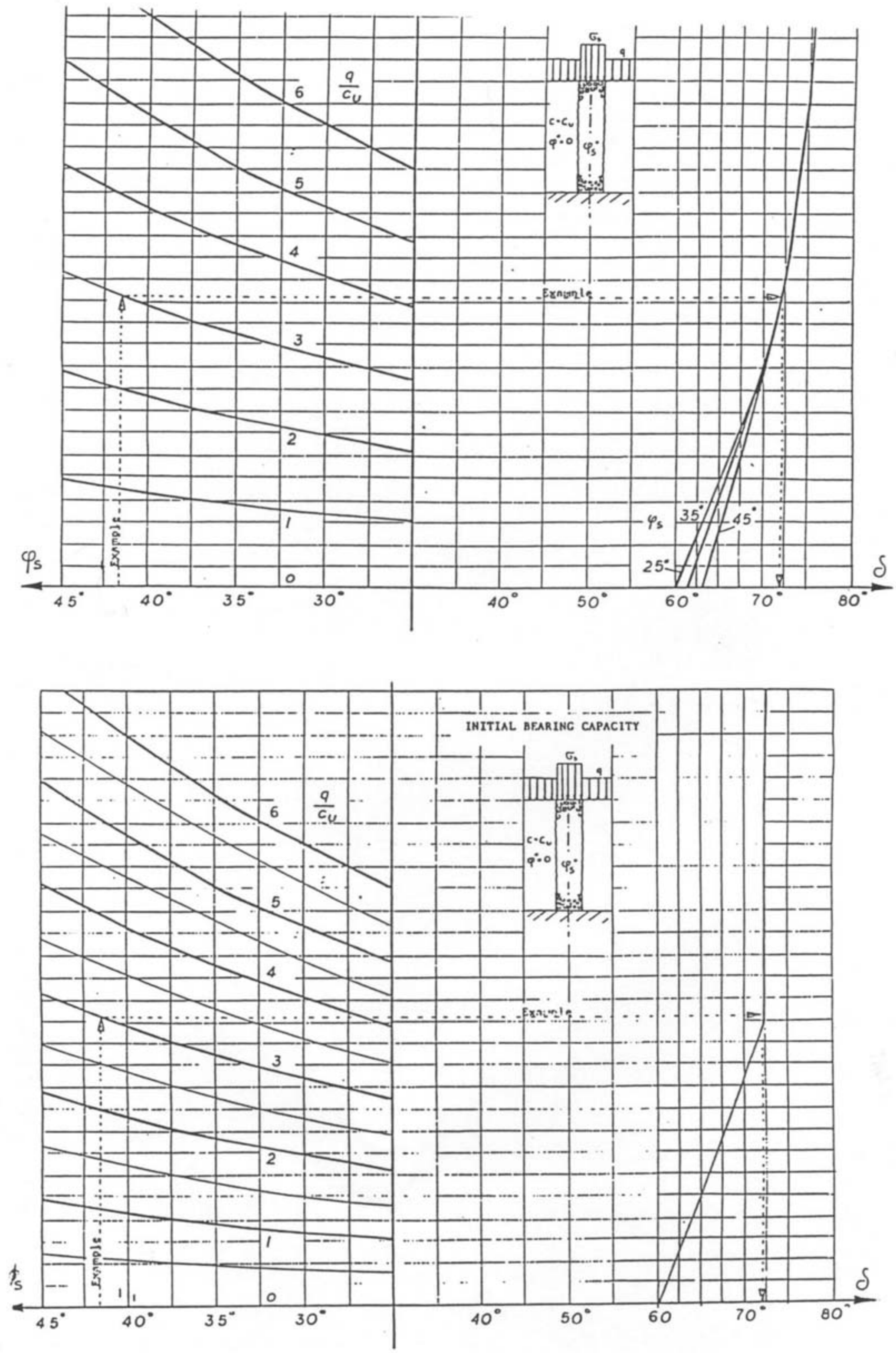


Figure 5.12 Values for δ When q > 0

(from: Hayward Baker Inc.)

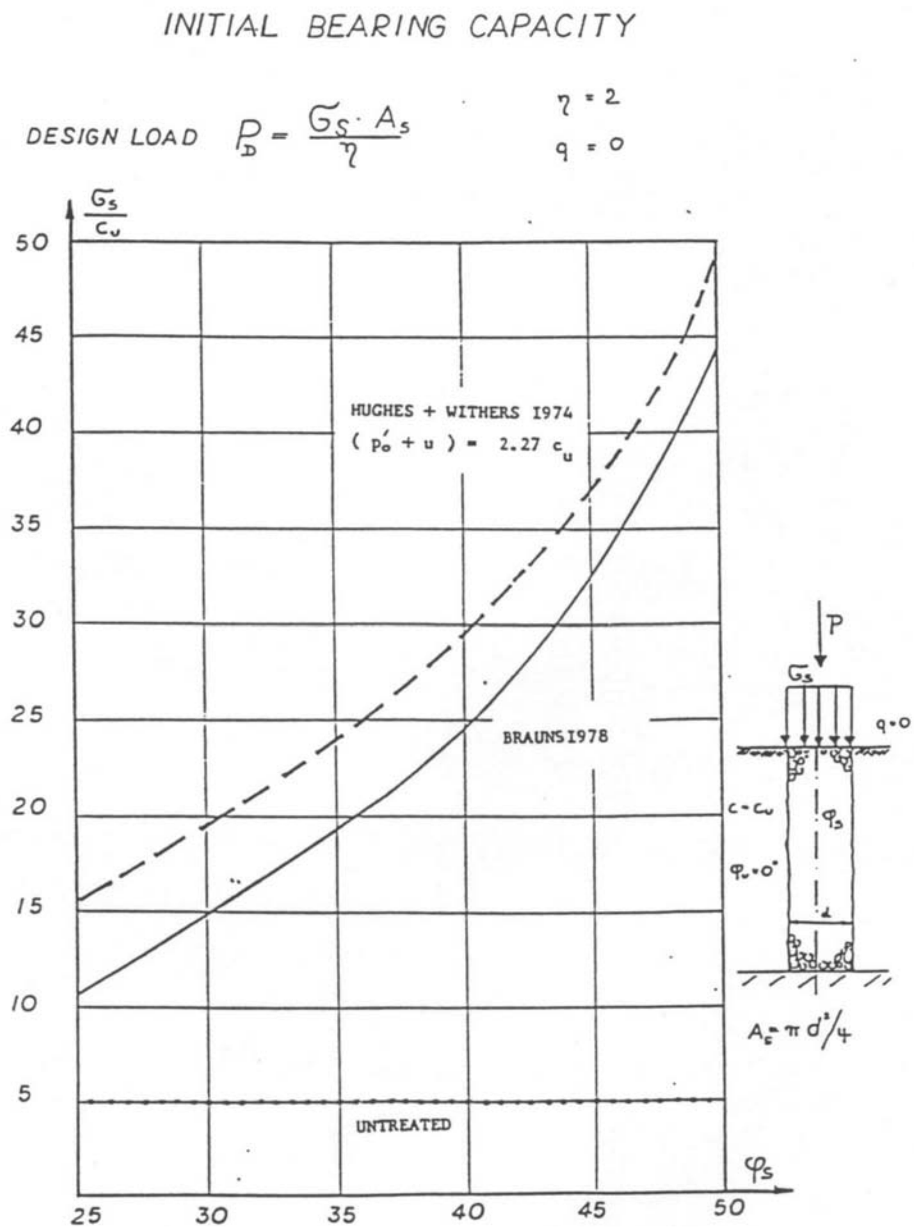


Figure 5.13 Values for δ When q = 0
(from: Hayward Baker, Inc.)

A) Bearing Capacity Factor Method

$$\sigma_v = \frac{\tilde{N}_{SC}\, C_u}{FS} = \frac{22\,(15\,\text{kPa})}{2} = 165\,\text{kPa}$$

$$\text{Column Capacity} = \sigma_v \times \text{Area} = 165 \ \text{kPa} \times \frac{\pi\,(1)^2}{4} = 130 \ \text{kN}$$

B) Brauns [Hayward-Baker]

$$\frac{\sigma_S}{c_u} = \left[\frac{q}{c_u} + \frac{2}{\sin 2\,\delta}\right] \bullet \left[1 + \frac{\tan \delta_S}{\tan \delta}\right] \tan^2 \delta_S$$

1) q (surcharge) = γZ = (17 kN/m^3)(0.2 m) = 3.4 kPa

q/c_u = 3.4/15 = 0.23

2) Using q/c_u = 0.34 from Figure 5.12 $\delta = 65°$ and $\delta_S = 45° + \phi/2 = 67.5$

3) $$\frac{\sigma_S}{c_u} = \left[0.23 + \frac{2}{\sin 2 \times 65°}\right] \bullet \left[1 + \frac{\tan 67.5}{\tan 65°}\right] \tan^2 67.5° = 35.2$$

4) Column Capacity = $$\frac{\sigma_S \cdot \text{Area}}{\text{FS}} = 35.2 \times 15\,\text{kPa} \times \frac{\pi\,(1)^2}{4} \times \frac{1}{2} = 207\,\text{kN}$$

Design load based upon 19 mm of settlement from a field load test was 94.3 kN. Thus for this example, both methods slightly over estimate the design bearing capacity.

Settlement

Reduction of settlement is one of the improvement benefits achieved by the use of stone columns. The reduction of settlement has been estimated by both pseudo-elastic and elastoplastic methods considering both isolated and wide spread loading using a unit cell concept (Barksdale and Bachus, 1983). The predicted improvement often expressed as the settlement ratio "n" defined as the ratio of settlement without stone columns to that with stone columns, is typically related to the area replacement (a_S) or area improvement ($1/a_S$) ratio. The settlement of the non-improved zone is determined by conventional settlement analyses.

Priebe [Haywood Baker] Method of Settlement

1. Obtain the compression index E_S from soil report or use values presented in Table 5.1.
2. Determine the footing area A.
3. Calculate the settlement s_u for the untreated soil conditions.
4. Determine the effective area ratio A/A_{se} for the footing according to Figure 5.14.
5. Obtain the improvement factor n from the Figure 5.11 using φ_S and A/A_{se}.
6. Calculate the expected final settlement $S = S_u/n$.

The effective area ratio A/A_{se} is based upon the number of supported sides within a stone column pattern. For example

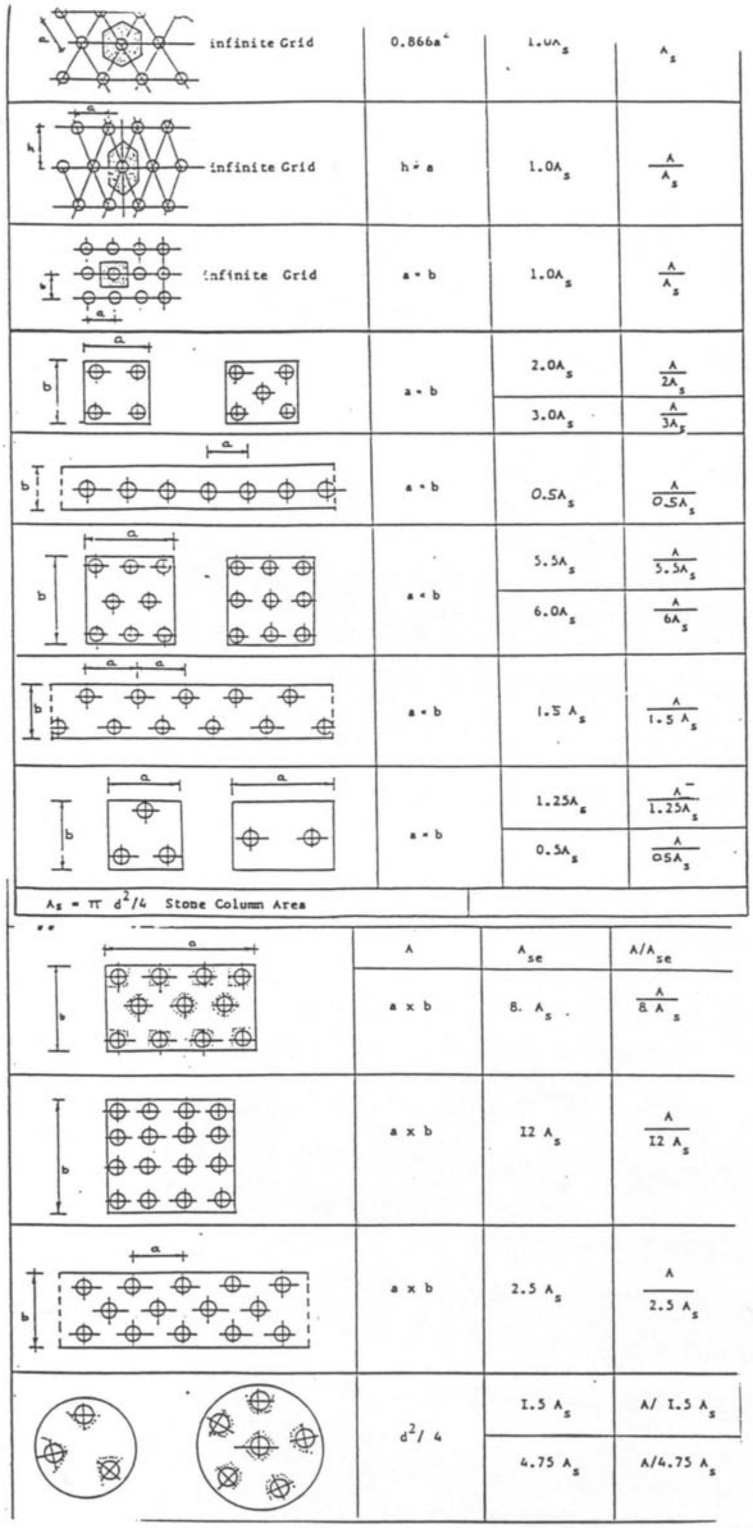
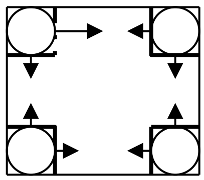


Figure 5.14 Examples of Effective Area Ratio, A/A_{SC}

(from: Hayward Baker, Inc.)

How to Obtain the Effective Area Ratio:



each stone column has four sides add up the supported sides

Example:

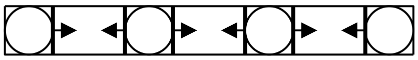
supported sides = 8

8 ÷ 4 = 2 stone columns full supported

A_S = area of stone column

effective area ratio $A/A_{se} = a/2A_S$

or



each stone column has 4 sides of which only 2 are supported = 2/4 = 0.5. Therefore the effective area ratio $A/A_{se} = A/0.5\ A_S$

Other examples are given in Figure 5.14.

The settlement improvement factor, η, is given as:

$$\eta = 1 + \tilde{A}\left[\frac{0.5 + F}{\tan^2\left(45 - \frac{\phi_S}{2}\right) \times F} - 1\right]$$

where $\tilde{A} = A_s / A$ and A_S = stone column area and A = unit cell from D_e

and $$F = \frac{1 - \mu^2}{1 - \mu - 2\mu^2} \times \frac{(1 - 2\mu)(1 - \tilde{A})}{1 - 2\mu + \tilde{A}}$$

Figure 5.15 graphically presents improvement factor values for various stone column friction angles and effective area ratio.

While there is no standard method for calculating settlements of stone columns, there are two techniques available: (1) convention consolidation with an improvement factor, and (2) use of finite element derived design charts (Hayward Baker, Inc.).

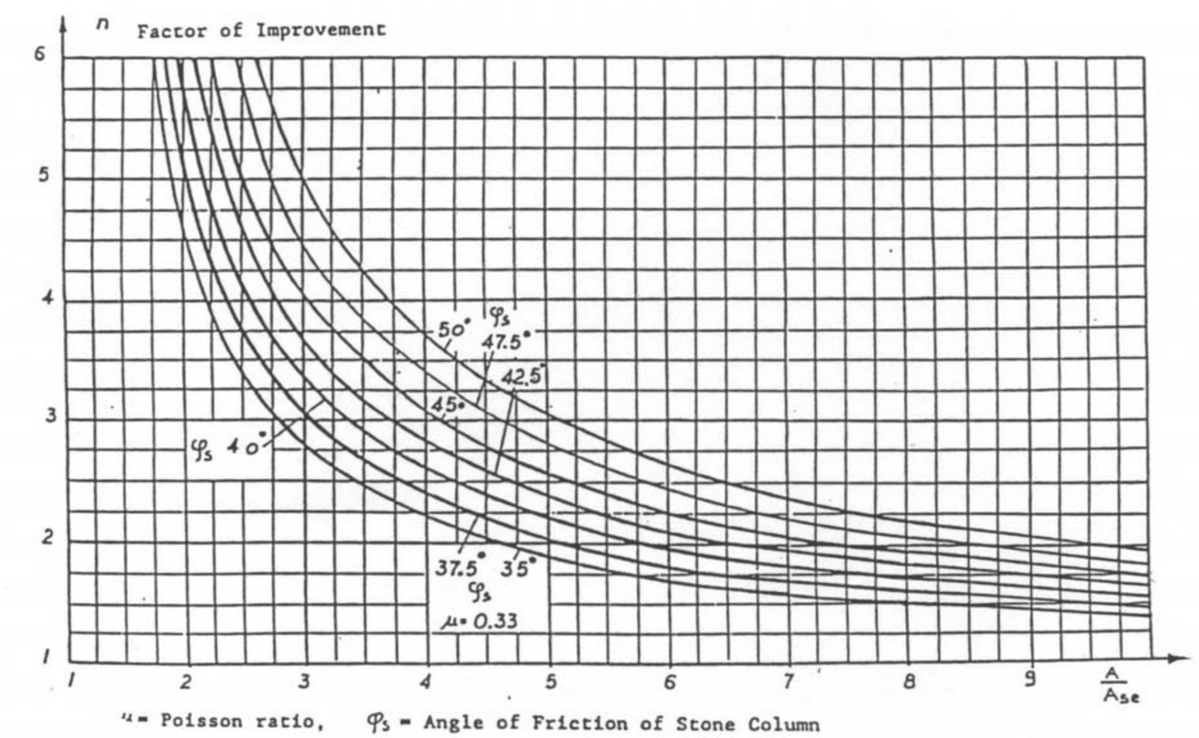
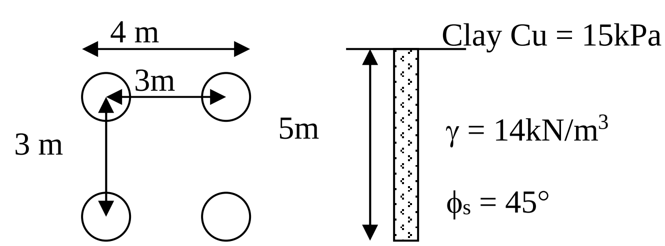


Figure 5.15 Settlement Improvement

(from: Priebe, 1976)

SETTLEMENT IMPROVEMENT (Priebe)

Priebe Settlement Example: Given: 1 m diam. stone columns on 3 m × 3 m center to center spacing. Estimate settlement for applied stress q = 250 kPa



1. Calculate the settlement improvement ratio, η assuming μ = 0.33

$$\eta = 1 + \tilde{A}\left[\frac{0.5 + F}{F\tan^2\left(45 - \frac{\phi}{2}\right)} - 1\right]$$

$$F = \frac{\left(1 - \mu^2\right)(1 - 2\mu)}{\left(1 - \mu - 2\mu^2\right)} \times \frac{1 - \tilde{A}}{1 - 2\mu + \tilde{A}}$$

A) $\tilde{A} = A_S / A$, total area, A = 3 m + 0.5 m + 0.5 m = 4 m × 4 m

$$A_S = \frac{\pi d^2}{4} = \frac{\pi (1)^2}{4} = 0.785\,m^2$$

$$\therefore \ \tilde{A} = 0.785/16 = 0.0491$$

B) $$F = \frac{(0.8911)(0.34)}{(0.4522)} \times \frac{(1 - 0.0491)}{(.3891)} = 1.6374$$

C) $$\eta = 1 + 0.0491\left[\frac{0.5 + 1.6374}{(1.6374)\tan^2 22.5^0} - 1\right] = 1.32$$

C-1 use Figures 5.14 and 5.15 to check A/A_{se} =

$$\frac{A}{2A_S} = \frac{16}{2(.785)} = 10.2, \quad \eta = 1.6$$

2. Estimate settlement if no stone columns present. Estimate E_S from Table 5.1 = 800 kPa

$$\delta = \frac{\sigma \ \square \ L}{E_S} = \frac{250\,kPa \times 5\,m}{800\,kPa} = 1.56 \ m$$

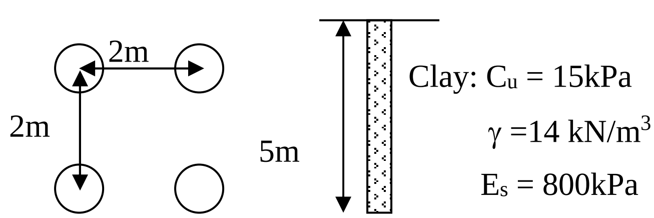
3. Settlement with Stone Columns

$$\delta = \frac{Suntreated}{\eta} = \frac{1.56\,m}{1.32} = 1.18 \ m$$

Bachus and Barksdale (1989) Settlement Example:

Given 1 m diam. Stone Columns on 2 m × 2 m square pattern.

Estimate Settlement for an applied stress q = 250 kkPa



A) $\alpha_S = A_S / A$

$$A_S = \frac{\pi (1)^2}{4} = .785 \ \text{m}^2$$

$A = 2 \text{ m} \times 4 \text{ m}^2$ $\qquad$ $\alpha_S = \frac{.785}{4} = 0.196$

B) $\sigma_C = \frac{q}{[1 + (\eta - 1)\, \alpha_S]} = \mu_C \, q$ $\qquad$ μ_C = stress concentration factor of clay

$\mu_C = \frac{1}{[1 + (\eta - 1)\, \alpha_S]}$ $\qquad$ using η recommend = 2.5 $\quad \mu_C = \frac{1}{1 + (2.5 - 1)(.196)} = 0.77$

C) $\mu_C = \frac{\delta_{treated}}{\delta_{untreated}}$

$$\delta_{untreated} = \frac{q \cdot L}{E_S} = \frac{(250)(5 \text{ m})}{800} = 1.56 \ \text{m}$$

$\therefore \ \delta_{treated} = \mu_C \, \delta_{untreated} = (0.77)(1.56 \text{ m}) = 1.21 \text{ m}$

D) Check using Bachus (1989) Figure 5.16.

1) $\alpha_S = 0.196$ and stress concentration factor, $\eta = 2.5$ gives $\mu_C = 0.77$ which checks step B.

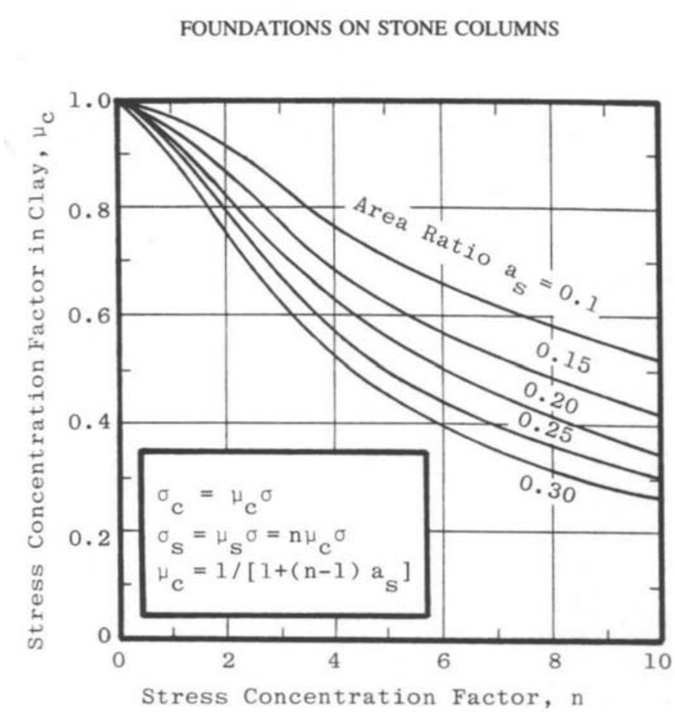


Figure 5.16 Variation of Stress Concentration Factor
(from: Barksdale and Bachus, 1983)

Time Rate of Settlement

Stone columns act similarly to wick drains in decreasing the distance which water has to flow in the radial direction for primary consolidation to occur.

Rate of Settlement

Stone columns substantially alter the time-rate of settlement as radial drainage governs. Therefore, time-rate of settlement computations are identical to the computations performed for sand drains. The effect of disturbance or smear during installation which reduces radial flow can be roughly accounted by reducing the diameter of the column by 50 to 80 percent of its design diameter. A larger disturbance or smear zone should be anticipated with the dry-displacement construction method and for all installations in sensitive clays (FHWA, 1999).

Shear Strength Increase

For slope stability analyses, an average shear strength of the soil stone column composite material is used along the sliding surface as shown on Figure 5.17 (Barkdale and Bachus, 1983).

The composite strength is a function of the undrained shear strength of the in-situ soil, the frictional resistance of the column, the area replacement ratio and the loading condition. For significant improvement to occur a relatively close spacing and a substantial overburden pressure is necessary to mobilize the frictional strength of the column (Greenward and Kirsch, 1984).

Average Shear Strength Method

The average shear strength method is widely used to analyze the stability of stone columns. In this method the weighted average material properties are calculated for the material within the unit cell. The soil having the fictitious weighted material properties is then used in a stability analysis. It is important to remember that stone columns must actually be located over the entire zone of material having weighted shear properties through which the circular arc passes. Since average properties can be readily calculated, this approach is appealing for both hand and computer usage. However, as discussed subsequently, average properties cannot in general be used in standard computer programs when stress concentration in the stone column is considered in the analysis (Barksdale and Bachus, 1983).

An example of a landslide problem involving stress concentration would be the use of stone columns to improve a soft soil to support an embankment. In this case the embankment surcharge produces a stress concentration in the stone column thereby increasing the resisting shear. This stress concentration can be easily handled via these equations (Barksdale and Bachus, 1983).

$$\sigma_C = \sigma/[1 + (\eta - 1)\,\alpha_S] = \mu_C\,\sigma$$

and $$\sigma_S = \eta\sigma/[1 + (\eta - 1)\,\alpha_S] = \mu_S\,\sigma$$

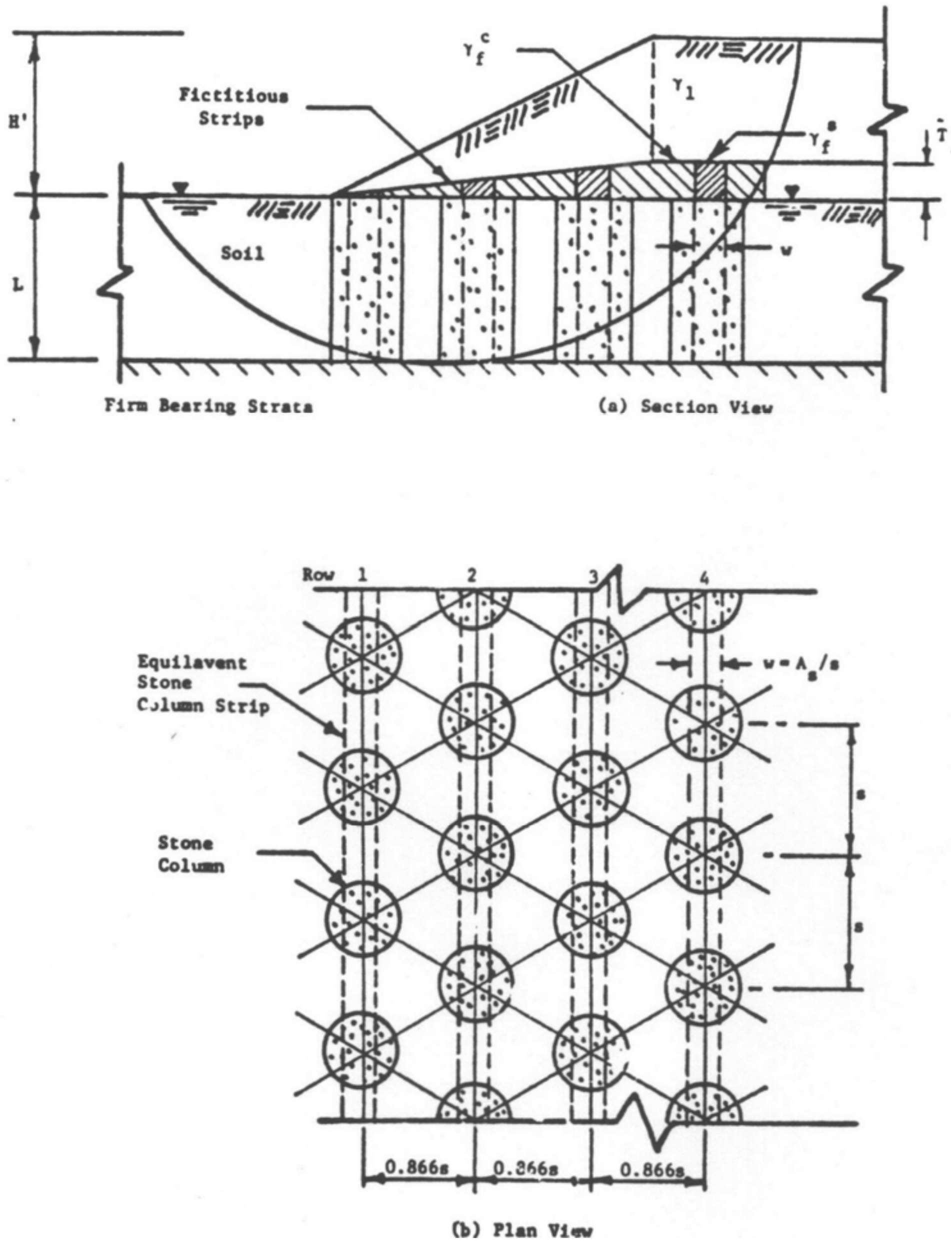


Figure 5.17 Slope Stability Analysis – Stone Column Strip Idealization and Fictitious Soil Layer (Frictionless Soil). (from: Barksdale and Bachus, 1983)

In the case of an embankment σ = the overburden stress and η is typically taken as 2.5 for design.

The average vertical stress, σ, acting at the interface of the embankment and stone column reinforced ground is usually assumed to be equal to the height of the embankment, H, times its unit weight γ_e. Accordingly let the stress concentration in the stone column be composed of 2 parts: σ and $\Delta\sigma_S$.

$\sigma_S = \sigma + \Delta\sigma_S$, where $\Delta\sigma_S$ is the stress that must be added to σ to give the correct stress concentration in the stone column. Thus rearranging the equation gives:

$$\Delta\sigma_S = \sigma_S - \sigma = \mu_S \sigma - \sigma = (\mu_S - 1)\,\sigma, \text{ which simplifies to:}$$

$$\Delta\sigma_S = (\mu_S - 1)\,(\gamma_e H)$$

Since the concept is to represent this stress concentration as an equivalent stress increase or

$\gamma_{fictitious} \times \sigma_{fictitious}$, by assuming $\sigma_{fictitious} = 1$ unit, then

$$\gamma_f^S = (\mu_S - 1)\,\gamma_e H$$

and $$\gamma_f^C = (\mu_C - 1)\,\gamma_e H$$

and $$\gamma_f^{ave} = \gamma_f^S \alpha_S + \gamma_f^C \bullet \alpha_C$$

If no stress concentration is present then, $\gamma_{average}$ is used as:

$$\gamma_{ave} = \gamma_S \alpha_S + \gamma_C \alpha_C$$

Obviously if the ground water table is present, then buoyant unit weights, γ', would be used accordingly.

Having determined the average unit weight, the shear strength parameters to use in the average shear stress method are:

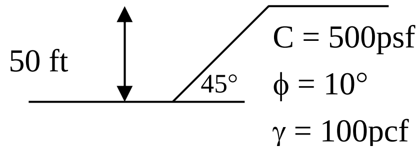
$$C_{ave} = C \bullet \alpha_C$$

$$[\tan\phi]_{ave} = \frac{\gamma_S \alpha_S \tan\phi_S + \gamma_C \alpha_C \tan\phi_C}{\gamma_{ave}}$$

An example is given as follows using a slope stability chart (Cousins, 1978). However a stability computer program can be as easily used.

Given: A 50 ft. high slope at 45° with C = 500 psf, ϕ = 10°, and γ = 100 pcf.

Find: Using Cousin's chart find the unreinforced FS, and subsequently the stone column spacing required for FS = 1.25

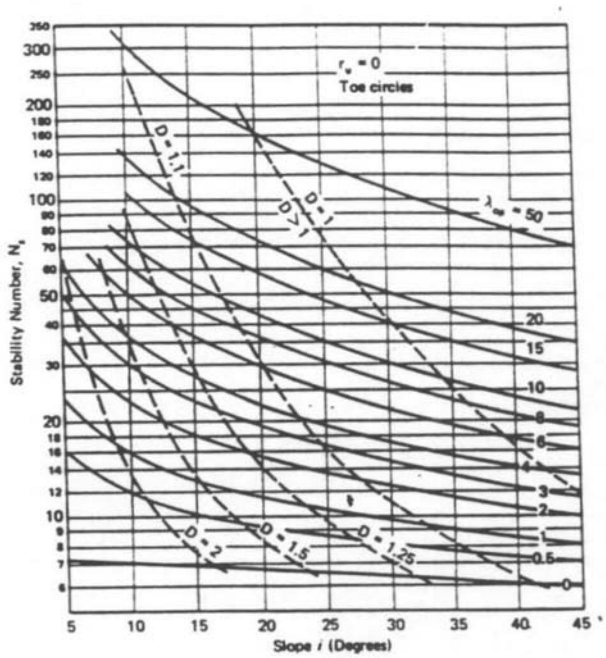


1. To use Cousin's chart (Figure 5.18) calculate

$$\lambda_{\phi C} = \frac{\gamma H \tan\phi}{C} = \frac{(100)(50)\tan 10°}{500} = 1.763$$

Using Cousin's chart $N_S = 9$

$$\therefore \quad FS = \frac{N_S C}{\gamma H} = \frac{9(500)}{(100)(50)} = 0.9 \quad \text{for untreated slope}$$



(b) Toe circle, $r_u = 0$

Figure 5.18 Cousin's (1978) Charts for Stability Number

2. What stone column spacing is required using a 3 ft-diameter stone column with ϕ_S = 45° placed in an equilateral triangular pattern to achieve a FS = 1.3?

a) $FS = \frac{N_S C}{\gamma H}$; $1.3 = \frac{N_S (500)}{(100)(50)}$; $N_S = 13$ and from chart $\lambda_{\phi C} = 3.8$ for 45° slope

b) $$\lambda_{\phi C} = \frac{\gamma_{ave} H \tan\phi_{ave}}{C_{ave}} = \gamma_{ave} \; H \frac{\left[\gamma_S \alpha_S \tan\phi_S + \gamma_C \alpha_C \tan\phi_C\right]}{\gamma_{ave}} \times \frac{1}{C \; \square \; \alpha_C}$$

but $\alpha_C = (1 - \alpha_S)$

so $\dfrac{\lambda_{\phi C}}{H} = \dfrac{\gamma_S \alpha_S \tan\phi_S + \gamma_C (1 - \alpha_S) \tan\phi_C}{C(1 - \alpha_S)}$

c) letting γ_S = 125 pcf, ϕ_S = 45°, γ_C = 100 pcf (given), ϕ_C = 10°, and C = 500 psf, H = 50

$\lambda_{\phi C}$ = 3.8 $\dfrac{3.8}{50} = \dfrac{125\alpha_S \tan 45° + 100(1-\alpha_S)\tan 10°}{500(1 - \alpha_S)}$

solving α_S = 0.14

d) for a triangular spacing pattern $\alpha_S = 0.907\ (D/S)^2$ and D = 3 ft diameter

$0.14 = 0.907\ (3/S)^2$

solving S = 7.6 ft

DESIGN VERIFICATION

As an important adjunct to design, a field verification program of load tests and in situ testing must be developed and implemented through appropriate construction specification requirements (Stack and Yacyshyn, 1991). A program should be specified, regardless of the contracting method (FHWA, 1999).

A combination of load tests on stone columns constructed before, during, and after production should be specified to verify the design assumptions and the performance specification. There are three types of load tests: (1) short-term tests which are used to evaluate ultimate stone column bearing capacity, (2) long-term tests which are used to measure the consolidation settlement characteristics, and (3) horizontal or composite shear tests which are used to evaluate the composite stone-soil shear strength for use in stability analyses. The most common of these tests is the short-term load test on a single column.

The short-term load tests similar to pile load tests should be performed after all excess pore pressures induced during construction have been dissipated. The load increment should closely correspond to the actual loading. For example, if the actual foundation load will be applied very slowly a load increment of approximately 10 percent of the ultimate should be used. A rapid loading may result in immediate settlement as well as consolidation settlement. If the actual load will be applied rapidly, a load increment of 20 to 25 percent of ultimate should be used. A final acceptance criteria of 2.5 cm of settlement at 150 to 200 percent of the allowable/design load appears to be a reasonable criterion.

The long-term settlement of the stone column foundation is usually estimated from the results of short-term load tests on single stone columns. Mitchell reported that the foundation settlement due to a uniform loading of a large area was 5 to 10 times greater than the settlement measured in a short-term load test on a single column. However, there is very little field data available to confirm this behavior. Therefore, it is recommended that long-term load tests on a group of columns be conducted in conjunction with short-term load tests to develop an estimate of the settlement of the stone column foundation. The long-term load tests should be conducted on a minimum of three to four stone columns located within a group of 9 to 12 columns having the proposed spacing and pattern. The load should be applied over the tributary area of the columns and left in place until the cohesive soil reaches a primary degree of consolidation of 90 to 95 percent. The applied load could consist of column backfill material, native material, and/or the dead weight from the short-term load tests. The results of these tests will provide valuable information for estimating the ultimate settlement of the stone column foundation.

During the production phase of construction, a few short-term load test can be performed for quality control purposes. These tests are referred to as proof tests and are used to verify quality control during production. The load applied in the proof test is usually 150 to 200 percent of the allowable/design load.

In-situ testing to evaluate the affect of the stone column construction on the native cohesive soil can be also specified. However, the specified test method should be selected on the basis of its ability to measure changes in lateral pressure in cohesive soils. The electric cone penetrometer (CPT), the flat plate dilatometer (DMT), and the pressuremeter (PMT) appear to provide the best means for measuring the change, if any, in lateral stress due to stone column construction. Due to the limited amount of information that will be obtained from CPT, DMT or PMT testing after column construction, it is recommended that long-term load tests on groups of stone columns be conducted instead of in-situ tests. However, extensive in-situ testing should be conducted during the initial subsurface investigation to reliably estimate the soil profile and the stone column design parameters.

Hayward-Baker (Tampa) routinely performs field load tests using a 10’ × 10’ steel plate secured on the corners with rock anchors. Testing follow as Tm D-1149 – Load testing for Driven Piles.

Mullins et al. (2000) has performed tests in conjunction with Hayward Baker using the 4 MN hydraulic catching system provided the opportunity to investigate the Statnamic response of shallow foundations on stone columns. This program consisted of side by side comparisons of full-scale Statnamic and static load tests on a 2 m square steel footing.

Shown in Figure 5.19 are the results of the Statnamic testing of the stone column shallow foundations, and the comparison of derived static and true static response of the footing.

As indicated in Figure 5.19, the Statnamic response of the stone column foundation was very similar to the true static response. Because neither test was conducted beyond the bearing capacity of the foundation, the response for both was highly linear. After what appears to be some initial device seating, the Statnamic-derived static load curve has a slope that is nearly identical to the true static. The apparent strain hardening observed in both methods was most likely due to the flexibility of the plate.

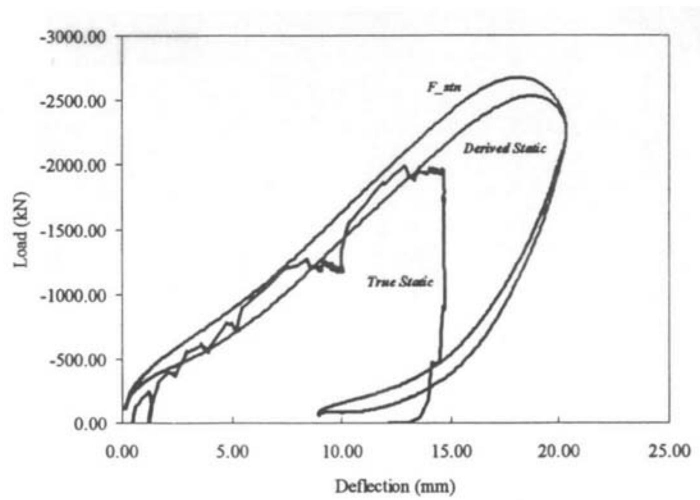


Figure 5.19 Comparison of Static and Statnamic Results for Shallow Foundations on Stone Columns (from: Mullins et al., 2000)

COST DATA

This section (FHWA, 1999) presents guidelines for preparing budget estimates in order that the economic feasibility of stone columns may be determined. There are many factors affecting the price of stone column construction including labor, the price and availability of stone, weather, environment, etc. Therefore, it is recommended that experienced contractors with a record of installing stone columns be contacted to verify both the budget cost calculations and the technical feasibility of stone column installation.

Basic Budget Estimate for Stone Column Installation

A budget estimate would typically include the following elements

Mobilization/Demobilization

Mobilization/demobilization costs will depend on the number of rigs required to complete the work on schedule, the type of crane needed to support the vibrators and the distance required for the equipment to be transported. As a minimum, the mobilization/demobilization cost will be $15,000 (1998) per rig.

Cost of Stone Column Installation

The basic cost of stone column installation is calculated by:

1) Calculating the number of stone columns required by dividing the square meter spacing determined for each stone column into the total treatment area.
2) Multiplying the number of stone columns by the depth of treatment.

The material cost of the stone backfill is a major component of the project and can account for over 40 percent of the estimated cost of stone column installation. The cost of stone backfill will vary from site to site. Where suitable backfill material is not readily available, the increased transportation costs may also affect the stone column installation pricing. Estimated costs in Florida for stone are $6 - $12 per 10 kN (ton).

The minimum cost for vibro replacement stone columns installation, *based on readily available suitable backfill material*, is $45 per linear meter, with a dry vibro displacement stone column starting at $60 per meter.

Cost Example

Estimate the cost for stone column installation to support a highway embankment 75 m wide by 300 m long placed over loose silty sand with a depth of 10 m. Stone columns are to be placed on a 3 m square grid pattern.

1. Each column would account for 9 sq. meters of treated area, equating to the installation of 2500 stone columns. Installation to 10 meters would equate to 25,000 linear meters of stone column.
2. Assuming a stone column diameter of 1 m, equates to 7.85 m^3 of stone required for each column. At a unit weight of 20 kN/m^3, 157 kN of stone is required per column. At a cost of $6/10 kN, the cost for stone is $94.20/column. Thus, for 2500 stone columns, the cost for stone will be $235,500.
3. Typical installation costs for vibro replacement stone columns are $30 per linear meter thus for 25,000 linear meters @ $30 per linear meter equates to $750,000.
4. Mobilization/demobilization costs are estimated at $15,000 per rig. Assuming 2 rigs are required for this project results in a cost of $30,000.
5. Total project cost is estimated as:

a)	cost of stone	$	235,000
b)	installation cost		750,000
c)	equipment cost		30,000
	Total	$	1,015,000

To this cost would be added, testing, inspection, and profit.

STONE COLUMN IMPROVED AND PIEZOCONE TESTED SITE SUPPORTS MID RISE BUILDING COMPLEX – A CASE HISTORY

Building Foundation Improvement – Ft. Myers, FL (Saxena and Hussin, 1997)

Hibiscus Pointe, a beachfront resort, is located off Estero Bay in For Myers Beach, Lee County, Florida. The building complex included 6-story and two, 5-story, over-parking buildings, along with recreational facilities.

A generalized subsurface profile compiled from 20 SPT borings is illustrated in Figure 5.20.

Figure 5.20 shows that in general the project site stratigraphy consisted of loose to medium dense, poorly-graded sand fill to approximately 2.5 m (7.5 ft) overlaying a 0.3 to 0.8 m (1.0 to 2.5 ft) thick zone of silty to sandy peat material. These strata are underlain by fine sands to silty fine sands (with some shell fragments) to a depth of 6 to 12 m (20 to 40 ft). Predominantly medium dense silty to sandy limerock (a mixture of silt, sand, clay, and gravel-size rock pieces) was encountered to a depth of 12 to 18 m (40 to 60 ft). Soft to very stiff silts to clays existed thereafter to the termination depth of 23 m (75 ft) in most of the test borings. The water table at the site ranged from 1.2 to 1.5 m (4 to 5 ft) below ground surface and was influenced by tidal fluctuation. Approximately 1.2 m (4.0 ft) of structural fill material was required to achieve the desired building grades.

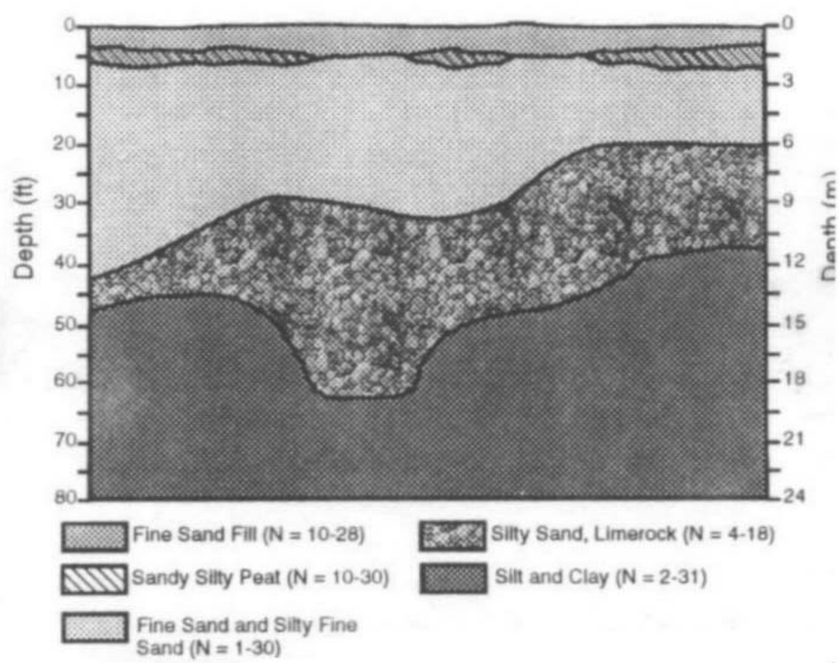


Figure 5.20 Subsurface Profile at Hibiscus Pointe (Saxena, 1997)

Foundation Design and Alternatives

The first step of the design process was to evaluate various foundation systems and associated costs, as shown in Table 5.2. Foundation alternatives considered included:

(i) driven displacement pilings - long and short

(ii) cast in-place auger piles, and

(iii) spread footings on ground improved by vibro-replacement (stone columns)

After a review of various foundation alternatives, the ground improvement technique of vibro-replacement for shallow ground improvement was proposed to partially replace the shallow zone of sandy peat and underlying silty fine sans encountered in the exploration to a depth of 6.2 to 8.5 m (20 to 28 ft) below the prepared and graded surface. The technique is feasible in peat as long as the thickness of the peat layer is not greater than the diameter of the vibro-replacement stone column. Vibro-replacement was recommended as a foundation design approach to achieve the goals of a higher design bearing pressure and a reduction of differential settlement, thereby allowing the use of a spread footing foundation system. In addition, a cost saving of as much as 25 to 30 percent could be realized if the vibro-replacement option was selected.

Table 5.2 Foundation Alternatives and Cost Comparison Summary

Item	Total No. of Stone Columns or Piles	Breakdown by Depth/Length	Total Footage & Unit Cost ($/m)	Stone Column or Pile Cost ($)	Mob./ Demob. Estimated ($)	Total Cost ($)
VIBRO-REPLACEMENT (STONE COLUMNS)						
Spread footings	115	48 @ 7.7 m,	742 m @ $39.40	29,235	2,500	31,735
PILE FOUNDATION ALTERNATE (SHORT PILES)						
Driven	110	9.8 m	1,078 m @ $39.40	42,475	2,500	44,975
Augercast	110	9.1 m	1,001 m @ $39.40	39,440	2,500	41,940
PILE FOUNDATION ALTERNATIVE (LONG PILES)						
Driven	75	15.2 m	1,140 @ $39.40	44,916	2,500	47,416
1. For short piles consider concrete or augercast piles. 9.8 and 9.1 m long rated for 270 kN.						
2. For long piles consider concrete piles. 35 cm square and 15.2 m long rated for 405 kN.						
3. All numbers are estimated.						

The settlements were estimated in general accordance with the method developed by Goughnour (1983) and Priebe (1993), as well as the assumption that the stone column and surrounding fine soils settle equally under the superimposed load.

The subsurface profile consisted of fine sand, peat and silty fine sand strata. The performance prediction anticipated that the fine sands would be densified to 70 percent relative density and the peat and silty fine sand strata would not densify between the stone columns. The performance improvement of the non-densifiable strata would be due to the reinforcement with the 1.1 m and 1.2 m (3.5 and 4 ft) diameter stone columns in the silty sands and peat, respectively. A standard settlement analysis was initially performed, accounting for the densification of the clean sands. The settlement, due to the peat and silty fine sand layers, was reduced accounting for the reinforcing effects of the stone columns. The post-vibro stone column settlement predictions for a standard foundation layout was in the range of 12 to 19 mm (0.5 to 0.75 in).

A design bearing pressure of 215 kPa (4,500 psf) correlating to a relative density criterion of 70 percent was selected (Saxena, 1987). The final foundation plan incorporated a minimum embedment bearing depth of 1.2 m (4.0 ft) for the exterior and interior spread footings and total dissipation of loading stresses within the densified depth of 6.1 to 9.1 m (20 to 30 ft).

Vibro-Replacement Program

For this project, a probe spacing of approximately 1.8 m (6.0 ft) was calculated to achieve the required improvement. Vibro-replacement stone columns were installed to firm soils encountered in the depth range of 6.1 to 9.1 m (20 to 30 ft) below the prepared ground surface. The vibro-replacement program for the project effectively utilized over 1,300 stone columns for a total linear footage of 9.6 km (31,700 lf), with an average of 0.77 m^3 (1.0 cu. yd) of stone backfill placed per linear meter.

Monitoring and Testing

In recent years advances have been made in ground improvement methods whereby in-situ testing can be used to immediately determine compliance with project specifications and revise/modify the construction procedures as necessary. Furthermore, testing during actual operation and post improvement verification by PCPT soundings provide a continuous profile of subsoils and stone columns.

Placement Monitoring

During the performance of the vibro-replacement, full time monitoring and logging was performed to confirm that the stone columns were constructed in the design locations and to observe and document important field installation parameters, including the rate of probe penetration, the amount of time the probe was held at the bottom of the treatment depth, the amount of time spent in compacting the stone columns, the amperage of the probe during compaction and the quantity of crushed stone used. A typical monitoring log is illustrated in Table 5.3.

Post Vibro-Replacement Quality Assurance Tests

Subsequent to completion of vibro-replacement operations and following a 48-hr pore pressure dissipation period, PCPT soundings were taken to obtain a post vibro-replacement continuous cone point resistance profile. The quality assurance program consisted of 80 soundings (10 in each building footprint) performed to a depth of 6. to 9.1 m (20 to 30 ft) with a electro static cone penetrometer rig equipped with pore pressure measuring capability (piezocone).

Table 5.3 Typical Vibro-Replacement Monitoring Log

Vibrator		Type: S Probe Weight: 5280						Length: 34 ft Size: 180 HP			
Approval Criteria Bearing Capacity: 4500 PSF						Relative Density: 70%					
Sequence No.	Plan Probe No.	Depth		Time				Loader Buckets of rock added	Qty. Rock (yds)	Delay Time	Re-marks
		Act.	Spec.	Start	Bottom Hold	Comp. Time	Total				
1	142	20′	20′	8:12	2 min.	8:40	28 min.	6	12	8 min.	200 A
2	148	18′	18′	8:40	2 min.	8:55	15 min.	4	8		200 A
3	147	18′	18′	8:55	2 min.	9:10	15 min.	4	8		200 A
4	148	20′	20′	9:10	2 min.	9:27	17 min.	5	10		200 A
5	145	20′	20′	9:28	2 min. 50 sec.	9:41	13 min.	4	8		200 A
6	144	20′	20′	10:03	2 min.	10:15	12 min.	3	6	12 min.	200 A
7	143	20′	20′	10:16	3 min.	10:37	21 min.	4	8	4 min.	200 A
8	141	31′	31′	10:39	4 min.	11:02	13 min.	5	10		200 A
9	139	31′	31′	11:03	3 min.	11:24	21 min.	6	12		200 A
10	140	31′	31′	11:24	3 min.	11:47	23 min.	5	12		200 A
DATE: _______________						INSPECTOR: ______________________________					

PCPTs were pushed down selected stone columns to test the stone backfill density. Refusal (90 plus percent relative density or point resistance in excess of 600 tsf) was often encountered on the cone penetrometer rig column. Attempts were then made to perform the test in between the stone columns until the appropriate depth was reached or refusal was encountered. Based upon these PCPT soundings, a 70 percent relative density criterion for an allowable bearing pressure of 215 kPa (4500 psf), was deemed to have been achieved. Settlement data from survey bench marks on building 8 revealed settlements on the order of 18 mm (0.7 inches).

Slope Stability Improvement (from: FHWA, 1999) -

NYDOT Route 22, Wadhams, NY (from: Seng and Ramsey, 1988)

Along a 67 m length of New York Route 22, slow, continuous movements over a 10 year period had created a constant and expensive maintenance problem. The failure extended from the centerline of the roadway into a swampy area 40 m downslope. Of particular importance in addressing slope stability improvement alternatives was that the site lay within the Adirondack State Park and within 15 m of a registered wetland. Selection of the method of improvement was therefore governed by environmental considerations.

Borings conducted at the failure location revealed a 3 m thick layer of silty clay overlying a 3 to 6 m layer of over-consolidated, soft, silty clay. Beneath this clay layer was a silty gravel in which an artesean head was encountered. Geotechnical laboratory tests showed that the liquidity index and activity of the clay were 1.0 and 0.5, respectively.

Three potential solutions were analyzed: stabilizing berm, shear key, and stone columns. Berm treatment would require additional right-of-way in the wetland area, and the shear key alternative would require a wide and deep supported excavation. Both options would be prohibitively expensive. Stone column installation by the dry, bottom-feed method was selected as being technically feasible, environmentally acceptable, and economically advantageous. The stone columns would be installed through the soft clays into the gravel layer to intercept the slip plane near the gravel/clay interface at a depth of 4.8 m.

A minimum 0.3 m thick drainage blanket was placed over the work area prior to stone column construction. Concern that column installation would alter the in-situ stresses of the soil and increase pore water pressure required careful evaluation of installation sequencing. As a result, columns were installed form the bottom of the slope up. The arching effect of the stone column/soil interaction would allow much of the downward driving fore to be carried by the columns, preventing additional soil from being displaced downhill. Excess pore water pressure would dissipate through the vertical drainage path provided by the columns.

Following a five-column test section to establish acceptance criteria, production columns were installed to a maximum depth of 10.7 m through the clay layer and into the silty gravel. During initial production, piezometer and inclinometer readings verified that pore pressure build-up was local and rapidly dissipated and that slope movement was minor. This allowed the installation sequence to be re-appraised. Subsequent columns were installed in a cost effective manner that still met acceptance criteria.

Prior to production, slope movement was measured at approximately 0.08 mm per day. Over the course of the stone column installation, total additional movement was 3.3 mm. As of May, 1995, some eight years after project completion, New York D.O.T. reported little to no movement recorded.

GUIDELINE SPECIFICATIONS FOR
GROUND IMPROVEMENT BY VIBRO-REPLACEMENT (VR 02)

1.0 SCOPE OF WORK

1.1 This section shall include but is not limited to providing all equipment, material, labor, supervision and related services to do all soil improvement by Vibro-Replacement as specified herein and as indicated in the drawings and other contract documents.

1.2 Soil improvement by Vibro-Replacement will be limited to the areas as specified and indicated on the project drawings.

2.0 BID REFERENCE

All Vibro-Replacement bid work shall be based upon the following: (i.e. foundation plans, discussions, soils reports, etc.)

3.0 SOIL IMPROVEMENT CRITERIA

Perform appropriate Vibro-Replacement with granular backfill material beneath all column foundations and load-bearing wall foundations to provide the following criteria upon successful completion of each.

3.1 Vibro-Replacement

3.1.1 An allowable soil bearing capacity of ______ pounds per square foot (psf) with a maximum total settlement of ___ inches.

3.1.2 Vibro-Replacement should be carried out to at least a depth equal to the zone of influence of the footing. The zone of influence is equal to twice the width of a column footing and four times the width of a wall footing measured from the bottom of the footing as shown on the drawings. Foundation soils should be improved to a <u>minimum</u> depth of 15 feet below the bottom of the foundations.

4.0 BACKFILL MATERIALS

4.1 Vibro-Replacement

Unless otherwise stated, stone shall be used for Vibro-Replacement. The backfill stone should consist of relatively hard, angular to subangular durable rock fragments, with the size of particles in the range of $^{1}/_{8}$ inch to 1-½ inches. The material to be used should be approved by the Engineer. A gradation meeting the No. 57 (ASTM C33) criteria is acceptable.

5.0 EQUIPMENT AND PROCEDURES

Specific equipment and procedural specifications are left to the vibro contractor performing the Vibro-Replacement work to achieve the specified criteria. However, the following minimum guidelines shall be used.

5.1 The specialty vibro contractor shall use an electric down hole vibrator capable of providing at least 160 HP of rated energy and a centrifugal force of 20 tons. An appropriate metering device should be provided at such a location that inspection of amperage build-up may be verified during the operation of the equipment. Metering device may be an ammeter directly indicating the performance of the vibrator tip of the eccentric. Complete equipment specifications should be submitted to the Engineer prior to commencement of the fieldwork.

5.2 The specialty vibro contractor's vibrator shall be a minimum of 16-inches in diameter and be capable of creating a stone columns with diameters of up to 42-inches.

5.3 The specialty vibro contractor shall have a minimum of five (5) years experience and shall provide documentation of successful job completion relevant to the specific application to this job.

5.4 After penetration to the required depth, the vibrator probe shall not be withdrawn more than 4.0 feet at any time unless the stone stops flowing to the bottom of the vibrator.

5.5 Redriving the vibrator into the treated depth shall be attempted at approximately 2.0 to 4.0 feet intervals to observe resistance to penetration and amperage build-up. During redriving, the vibrator tip shall penetrate to within 2 feet of the previous redriving depth.

5.6 Amperage build-up and backfill quantities will be contingent upon the type of vibrator used and procedures. Prior to commencement of work, the contractor shall discuss the equipment capabilities with the Engineer to determine if trial probes will be necessary.

6.0 TESTING AND INSPECTION

(if STP testing to be performed, use first 6.1 through 6.6, if load test to be performed, use second 6.1 through 6.6)

6.1 All testing to determine specification compliance will be provided by the engineer, paid by the Owner, and will consist of Standard Penetration Test (SPT) borings and on-site observation during the Vibro-Replacement.

6.2 SPT borings shall be performed at locations at points which are equidistant and farthest from two, three, or four probes, whichever is applicable, with a multiple probe pattern under a column or wall footing.

6.3 The work should consider an averaging of SPT values for the test hole, and also the location of the test hole within the treated area. The required criteria for the production of this work should relate to parameters used in the stone column design (e.g. to limit settlement to __-inches and to provide a bearing capacity of ____ psf, etc.) as determined by the SPT work.

6.4 Areas of the site not adequately densified as determined by the specified SPT values shall be recompacted by Vibro-Replacement at no additional cost to the Owner.

6.5 The engineer will provide site inspection to insure performance of the Vibro-Replacement work. This inspection may include the following: observance of the vibro contractor's procedures, recording of backfill quantities, and recording of ammeter information.

6.6 A sample of the type of backfill material should be submitted to the engineer for a grain size distribution analysis to establish the suitability, the cost of which will be borne by the owner.

Or

6.1 Testing to determine specification compliance will be provided by the Contractor, and will consist of a Static Load Test.

6.2 The load test shall be performed at an actual foundation location chosen by the Engineer. The foundation area to be tested should measure 10 ft x 10 ft.

6.3 The load test shall be erected and performed for by the contractor at contractor's expense. The owner shall pay for all costs associated with monitoring of the test by the Engineer.

6.4 The load test shall be performed in general accordance with ASTM D-1143 Quick Test. The test foundation shall be loaded to 1.5 times the design load of _____ psf.

Settlements of the test foundation shall be measured halfway between the center and each of the four corners. The average of the four readings shall be used to confirm acceptance of the required settlement criteria. The contractor shall submit load test detail and setup.

6.5 The Engineer will provide site inspection to insure performance of the Vibro-Replacement work. This inspection may include the following: observance of the vibro contractor's procedures, recording of backfill quantities, and recording of ammeter information.

6.6 A sample of the type of backfill material should be submitted to the engineer for a grain size distribution analysis to establish the suitability, the cost of which will be borne by the owner.

7.0 LAYOUT OF WORK

After the initial rough grading of the vibro-treatment area, the general contractor shall accurately lay out center of foundations for the vibro contractor to locate vibro points, as shown on the pattern drawing furnished by the vibro contractor.

8.0 SUBMITTALS

8.1 The vibro contractor shall furnish a shop drawing to the geotechnical engineer (for review) prior to the work indicting the spacing, location and depth of the vibro points to achieve the criteria outlined in this specification.

8.2 A daily log shall be submitted to the Owner by the vibro contractor to include recording of probe number, start/finish time of probe, depth of treatment, approximate backfill quantities and indication of relative ammeter increases.

8.3 Any change in the predetermined vibro program necessitated by a change in the subsurface conditions will be immediately reported and submitted to the Engineer.

8.4 Load test detail and setup.

9.0 COMPLETION REPORT

Upon completion of the Vibro-Replacement work, the Engineer would prepare a report to the Owner documenting the observations and results of the tests. This report will certify that the bearing pressure has been achieved.

REFERENCES

ASCE (1987). Soil Improvement – Ten Year Update, GSP 12, edited by J.P. Walsh.

ASCE GSP No. 69, Geologan 476-491.

Bachus, R.C. and Barksdale, R.D. (1989). Design Methodology for Foundations in Stone Columns" Proc Foundation Engineering. Current Principles and Practices. ASCE, Vol. 1, pp. 244-257, Evanston, IL.

Baez, J.I. (1995). "A Design Model for the Reduction of Liquefaction by Vibro-Stone Columns" Ph.D. Dissertation, Univ. of Southern California, Los Angles, CA, 207 pages.

Barksdale, R.D. and Bachus, R.C. (1983). "Design and Construction of Stone Columns – Vol. 1" Report No FHWA/RD-83/02C. Federal Highway Administration, Washington DC.

Bauman, V. and Bauer, G.E.A. (1974). "The Performance of Foundations in Various Soils Stabilized by the Vibro-Compaction Method" Canadian Geotechnial Journal (11) 4 509-530.

Besancon, G., Iorio, J.P., and Soyez, B. (1984) "Analyse des Parametres de Calcul Intervenant dans le Dimensionnement des Colonnes Ballastees" Int. Conf. in In-Situ Soil and Rock Reinforcement, Paris 119-126.

Brauns, J. (1978). "Initial Bearing Capacity of Stone Columns and Sand Piles" Reinforcing and Stabilizing Techniques, Sydney, Australia.

Campanella, R.G. and Robertson, P.K. (1989). Guidelines *for Use and Interpretation of the Electronic Cone Penetration Test*, Soil Mechanics Series, University of British Columbia, Vancouver, Canada.

Cousins, B.F. (1978). "Stability Charts for Simple Earth Slopes," Jrn. of the Geotechnical Engr. Div. ASCE Vol. 104, No. GT2 267-279.

Engelhardt, K and Golding, H.C. (1975). "Field Testing to Evaluate Stone Column Performance in a Seismic Area" Geotechnique (25) 1.

FHWA (1999). Ground Improvement Technical Summaries Demonstration Project 116, Vol. 1 and 2, Publication No. FHWA-SA-98-086, Authors: Elias, V., Welsh, J., Warren, J. and Lukas, R.

Gaughnour, R.R. (1997). "Stone Column Construction by the Rotary Method" ASCE GSP No. 69, Ground Improvement Ground Reinforcement Ground Treatment, Geologen P. 492-506.

Gaughnour, R.R. (1983). "Settlement of Vertically Loaded Stone Columns in Soft Ground," Proceedings, Eighth European Conference on Soil Mechanics and Foundation Engineering, Helsinki, Finland, 235-240.

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

Hayward Baker Inc. – personal communication.

Hughes, J.M.O., Wither, A.J. and Greenwood, D.A. (1976), "A Field Trial of the Reinforcing Effects of a Stone Column in Soil," Ground Treatment by Deep Compaction, Inst. of Civil Engineers, London.

Hussin, J.D. and S. Ali (1987). "Soil Improvement at the Trident Submarine Facility," Proceedings of Soil Improvement-A 10 year Update, (12), 215-231.

Jebe, W. and Bartels, K. (1983). "The Development of Compaction Methods with Vibrators from 1976 to 1982" Eighth European Conf. on Soil Mechanics and Foundation Engineering, A.A. Balkema, Helsinki.

Mattes, N.S. and Poulos, H.G. (1969). "Settlement of a Single Compressible Pile," Jrn. of S M and Fdn. Engr., ASCE SM1, Jan.

Mitchell, J.K. (1981). "State-of-the-Art Report on Soil Improvement," 10th Int. Conf. in Soil Mechanics and Foundation Engineering, Stockholm.

Mullins, G., Garbin, E.J., and Lewis, C. (2000). "Statnamic Testing: University of South Florida Research" Proc. 2nd Int. Statnamic Seminar, STATNAMIC Loading Test, Balkema, p. 117-132.

Munfakh, G.A. (1984), "Soil Reinforcement by Stone Columns – Verified Case Applications," International Conf. on In-Situ Soil and Rock Reinforcement, Paris, 157-162.

Munfakh, G.A., Abramson, L.W., Berksdale, R.D., and Juran, I. (1987). "In-Situ Ground Reinforcement," Soil Improvement-A Ten Year Update, (12), 1-17.

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

Priebe, H.J. (1993). "Design Criteria for Ground Improvement by Stone Columns," Proceedings of the Fourth National Conference on Ground Improvement, Lahore, Pakistan.

Priebe, H.J. (1990). "Vibro Replacement – Design Criteria and Quality Control," Proceedings of Symposium on Design, Construction, and testing of Deep Foundation Improvement, ASTM, STP 1089, Las Vegas, Nevada.

Priebe, H. (1976). "Abschatzung des Setzungsverhaltens eins durch Stopfuerdichtung verbessertan Baugrundes" Die Bautechnick, H.S.

Saxena, D.S. (1987). "Piezocone – Quality Control Tool for Densified Sands," Proceedings, VII Pan American Conference, Soil Mechanics and Foundation Engineering, Cartagena, Columbia, Vol. 3, 373-384.

Saxena, Anupm, and Saxena, D.S. (1995). "Vibro-Replacement and PCPT for Site Improvement – A Case History," ASCE Annual Convention, San Diego, California.

Saxena, D.S. and Hussin, J.D. (1997). "Stone Column Improved and Piezocone Tested Site Supports Mid-Rise Building Complex – A Case History" Ground Improvement, Ground Reinforcement, Ground Treatment.

Schmertmann, J.H. (1978). Guidelines for *Cone Penetration Test Performance and Design*, FHWA Report No. TS-78-209, Federal Highway Administration, Washington, D.C.

Seng, J.T. and Ramsey, I.S. (1988). "Slope Stabilization by Stone at Wadhams, NY," Report by Soil Mechanics Bureau, NYDOT, State Campus, Albany, NY.

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

Wissa, Anwar, E.Z., and Fuleihan, N.F. (1983). "Piezocone Testing, Research, Theory and Applications," New Methods in In-Situ Testing Workshop/Seminar Sponsored by The University of Florida, Continuing Education Program, Orlando, Florida.

Chapter 6

SOIL–LIME, SOIL–CEMENT, SOIL–ASPHALT STABILIZATION

INTRODUCTION

Much as the ancient alchemists sought the philosopher's stone, a mysterious, unknown substance which they believed to have the power to transmute base metals into gold, so have soil engineers examined a myriad of chemicals to improve soil behavior. The purpose of mixing these chemical additives, such as lime, Portland cement, bitumen, calcium chloride, calcium sulfate (gypsum), sodium hydroxide, etc., with soil is to:

1. Increase strength
2. Plasticity reduction
3. Drying action
4. Reduction in swelling potential
5. Improve bearing capacity
6. Reduce settlement
7. Reduce permeability
8. Reduce erosion

Despite the myriad of chemical tested for soil stabilization, the most popular three additives are: lime, Portland cement, bitumen, (lime-flyash is a combination).

LIME MODIFICATION vs. STABILIZATION

Modification is the mixing of small (< 3%) amounts of additive producing immediate changes in physical behavior; i.e., reduction in plasticity, drying action. Modification is usually employed to develop working platforms in soft ground situations.

Stabilization is different than *modification* and is the mixing of sufficient additive (5%-8%), sufficient to develop long-term strength improvements. Stabilization can continue for a very long period of time as long as sufficient lime is present and the pH remains high (above 10) Stabilization can result in significant strength gains (by a factor of 20+ in some cases) is usually employed for roadway subbase improvement, and deep soil mixing.

SOIL–LIME AND SOIL–CEMENT CHEMICAL REACTIONS

The “key” to understanding soil-lime and soil-cement stabilization in an understanding of the chemical reaction involved. The reaction of lime, water, and soil forms cementatious products of calcium – silicate – hydrates (CSH) and calcium – aluminates – hydrates (CAH), which cement the particles together. Quicklime (CaO) when hydrated (H_2O) forms calcium hydroxide [$Ca(OH)_2$] or slaked lime, which has a pH ≈ 12.4. This slaking also produces about 65.3 kJ/mol of heat for drying action.

$$CaO + H_2O \rightarrow Ca(OH)_2 + \text{heat} \ (65 \text{ kJ/mol})$$

Remembering that clay minerals are complex alumino-phyllosilicates composed of two basic units: the silicon tetrahedron and the aluminum octahedron, and possess a net negative charge due to isomorphic substitution, cation exchange occurs immediately when lime is mixed with soil. Clay minerals are generally considered in three relatively common and distinct groups namely kaolinite, illite and montmorillonite. Each of these minerals possesses different reactivity to lime modification, with kaolinite being least reactive due to its low ion exchange capacity and largest particle size, whereas montmorillonite is the most reactive due to its highest exchange capacity and smallest size.

The consequences of soil-lime mixing are:

a. Hydration – The reaction of lime (CaO) with water causes generation of heat, which in turn dries the soil.

b. Flocculation and Agglomeration – Flocculation occurs due to the higher valence Ca^{++} cation displacing lower valance ions from the clay surface. This displacement, in turn, depresses the double water layer surrounding the clay platelets allowing the clay particles to flocculate and agglomerate. This agglomeration causes the agglomerated clay flocs to function as silt size particles, resulting in reduced plasticity and increased workability of the clay soil. The soil permeability is improved for drainage (but also more erodeable) establishment of a working platform. Usually small amounts (<3%) are required to achieve these changes and have been termed the “lime fixation percentage” or modification percentage.

c. Pozzolanic Reactions – The Romans (circa 300 BC) found that by mixing a pink sand-like material from the town of Pozzuoli with their normal lime they obtained a stronger cement material. The pink sand turned out to be fine volcanic ash and they had inadvertently produced the first 'pozzolanic' cement, named for the town, Pozzuoli. Pozzolanic reactions are by definition, any siliceous or siliceous and aluminous material which possesses little or no cementitious value in itself but will, if finely divided and mixed with water, chemically react with calcium hydroxide to form cementatious C-S-H and C-A-H compounds.

   In a soil-lime-water mixture, the pH increases rapidly due to the partial dissociation of the calcium hydroxide from the lime until a pH of 12.3 is reached. In this highly basic medium, the silica from the clay silica tetrahedral increases in solubility (400 ppm at pH 10 to 5000 ppm at pH 11) the negative charge possessed by the clay particles also greatly increases in this alkaline medium. The opposing charges cause

> the highly electropositive calcium ions to be readily adsorbed by the clay particles, neutralizing the clays permitting closer contact until flocculation occurs. (Ho et. al., 1963, and Eades, 1962) At points of contact within the clay flocs limited chemical reaction products ‘spot weld” the contact points. If the presence of sufficient calcium is maintained, i.e., more than the “modification percentage,” pozzolanic reactions slowly occur. The severe attack on the silica tetrahedral by the highly alkaline water partially decomposes and alters the clay mineral structure liberating silica and alumina for the formation of these insoluble cementatious C-S-H and C-A-H compounds. Following compaction, which forces the particles closer these reaction products slowly gain crystallinity and stabilize the soil.

Hence successful soil –lime stabilization requires a source of silica and/or alumina, a high basic condition, small particles susceptible to cation exchange, and time. Sand, for example, although possessing a source of silica, does not react favorably with lime due to its large particle size and lack of cation exchange capacity. Likewise, acid, or organic soils do not react favorably to lime stabilization. Thompson (1964) reported that soils containing more than 1% organics usually do not respond well to lime treatment. Hence lime treatment of surface soils is not encouraged.

It has been reported (Bredenkamp and Lytton, 1994) that clays high in sulfates can cause a lime stabilization problem because the calcium in the lime may react with the clay and the sulfates, and form expandable minerals like ettringite, which in turn, can expand to 200 percent of its original size and create heave/swelling problems. This resulting expansion creates cracking of roadways.

SOIL–PORTLAND CEMENT REACTIONS

Soil – PC reactions are quite similar to soil –lime reactions in that the stabilization is the result of the formation of C-S-H and C-A-H compounds. Portland cement manufacturing blends a source of calcium (usually limestone), and silicates with aluminates (usually clay). The raw materials are ground and mixed, then roasted in a rotary cement kiln up to 1480º C (2700º F). Two reactions occur. First, the limestone or calcium carbonate turns into lime and carbon dioxide. Then the lime combines with the silicates to make dicalcium silicate (25%) and tricalcium silicate (55%) and with the aluminates to make tricalcium aluminate (10%) and tretracalcium aluminoferrite (8%) . This is cooled and the resulting clinker is ground to a fine powder, resulting in portland cement. Upon the addition of water, the two silicates hydrate to form C-S-H , which provides most of the strength. The tricalcium aluminate also hydrates, but contributes little to the strength, but is the source of the gray color.

Hence successful soil – cement stabilization is quite similar to soil – lime stabilization, except that the cement provides lime and a source of silica. Acid soils, organics, and sulfates are detrimental to soil –cement stabilization.

LIME – FLYASH AND CEMENT – FLYASH REACTIONS

Flyash is a solid waste product created by the burning of coal at electrical power generating plants. It ascends the chimney; i.e., "flies", where it is captured via electrostatic precipators, filters, etc. Flyash is a pozzolan. Consequently, it is an excellent source of fine-grained silica for stabilization.

MIX DESIGN

Figure 6.1 presents the Corps of Engineers gradation triangle for aid in selecting lime, Portland cement, or bituminous stabilizer, as well as approximate lime contents as suggested by Ingles and Metcalf (1973). As shown, lime is recommended for soils containing clays or restricted to soils having a PI> 12 and more than 5% fines. These requirements are designed to provide clays and sufficient plasticity so as to be reactive.

Portland cement, as expected is recommended primarily for cohesionless soils, but can be used in clayey soils. For cohesionless soils restrictions on Portland cement are: PI < 30 or PI < $20+\frac{50-\textit{fines content}(\%)}{4}$. These restrictions pertain to ease in mixing PC + soil, and excessive clay presence interfering with PC – coarse-grained particle reaction. Two stage treatments of using lime to diminish the clay effects followed by PC application are possible, but expensive. In the case of PC + cohesive soils, a LL < 40 and PI < 20 is recommended as clay plasticity (workability) interferes with efficient mixing of the PC + soil.

Bituminous stabilization is recommended for cohesionless soils with less than 30% fines and PI's < 10. These restrictions are again a mixing issue, as clay plasticity interferes with efficient mixing of bitumen + soil. Lime treatment to neutralize clay plasticity prior to the application of bitumen is feasible, but expensive.

Mix Design Procedures for Soil—Lime Mixtures

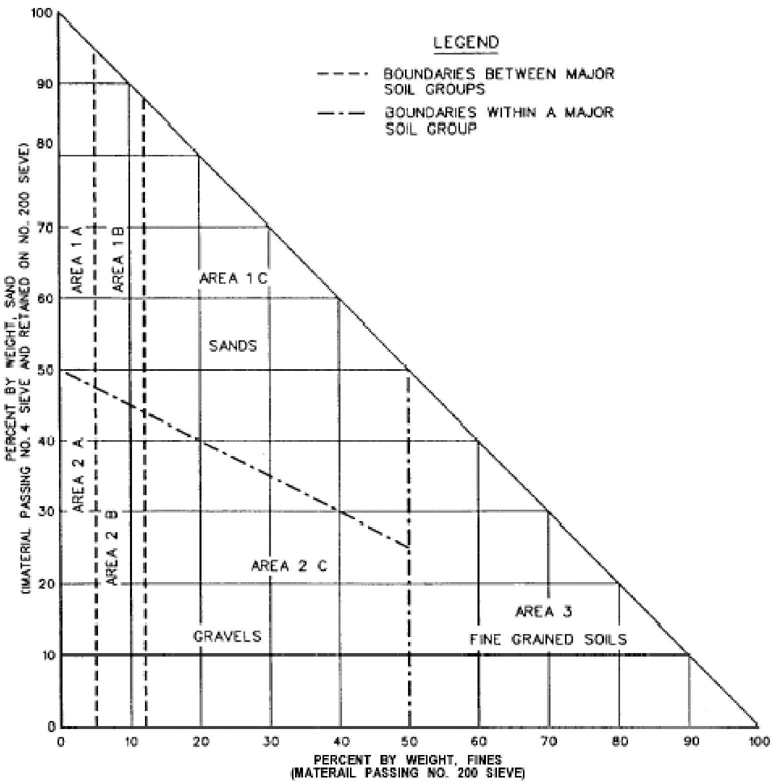
Of the myriad of mix design procedures, the Air Force Soil Stabilization Index System (SSIS) (Epps, and Dunlap, 1971, Dunlap, 1975) incorporates the best features and state of art of several design procedures. The SSIS subsystem for non-expedient subgrade stabilization with lime is presented in Figure 6.2. The procedure consists of four steps in selecting the optimum lime content (OMC), specifically:

a. Use pH test of Eades and Grim (1966) to estimate approximate lime contents. (Appendix 6.A)

b. Determine the lime reactivity (strength) via unconfined compression tests (U/C) of several soil—lime mixtures using Thompson's (1964) criteria of a 50-psi (3.6 tsf), ΔUCT > 50 psi, strength increase to assess reactivity.

c. Determine the durability via U/C strength after a 24-hr immersion, and apply Biswass' (1972) criteria of an immersed strength of 30 psi (2.16 tsf).

d. Select the optimum lime content (lowest percentage meeting these criterion).



Area	Soil Class[a]	Type of Stabilizing Additive Recommended	Restriction on LL and PI of Soil	Restriction on Percent Passing No. 200 Sieve[a]	Remarks
1A	SW or SP	(1) Bituminous			
		(2) Portland cement			
		(3) Lime-cement-fly ash	PI not to exceed 25		
1B	SW-SM or SP-SM or SW-SC or SP-SC	(1) Bituminous	PI not to exceed 10		
		(2) Portland cement	PI not to exceed 30		
		(3) Lime	PI not to exceed 12		
		(4) Lime-cement-fly ash	PI not to exceed 25		
1C	SM or SC or SM-SC	(1) Bituminous	PI not to exceed 10	Not to exceed 30% by weight	
		(2) Portland cement	--[b]		
		(3) Lime	PI not less than 12		
		(4) Lime-cement-fly ash	PI not to exceed 25		
2A	GW or GP	(1) Bituminous			Well-graded material only
		(2) Portland cement			Material should contain at least 45% by weight of material passing No. 4 sieve
		(3) Lime-cement-fly ash	PI not to exceed 25		
2B	GW-GM or GP-GM or GW-GC or GP-GC	(1) Bituminous	PI not to exceed 10		Well-graded material only
		(2) Portland cement	PI not to exceed 30		Material should contain at least 45% by weight of material passing No. 4 sieve
		(3) Lime	PI not less than 12		
		(4) Lime-cement-fly ash	PI not to exceed 25		
2C	GM or GC or GM-GC	(1) Bituminous	PI not to exceed 10	Not to exceed 30% by weight	Well-graded material only
		(2) Portland cement	--[b]		Material should contain at least 45% by weight of material passing No. 4 sieve
		(3) Lime	PI not less than 12		
		(4) Lime-cement-fly ash	PI not to exceed 25		
3	CH or CL or MH or ML or OH or OL or ML-CL	(1) Portland	LL less than 40 and PI less than 20		Organic and strongly acid soils falling within this area are not susceptible to stabilization by ordinary means
		(2) Lime	PI not less than 12		

[a] Soil classification corresponds to MIL-STD-619B. Restriction on liquid (LL) and plasticity index (PI) is in accordance with Method 103 in MIL-STD-621A.

[b] $PI \leq 20 + \frac{50 - \text{percent passing No. 200 sieve}}{4}$

Figure 6.1 Gradation Triangle for Aid in Selecting Stabilizing Agent (from ARMY TM 5-822-14,1994)

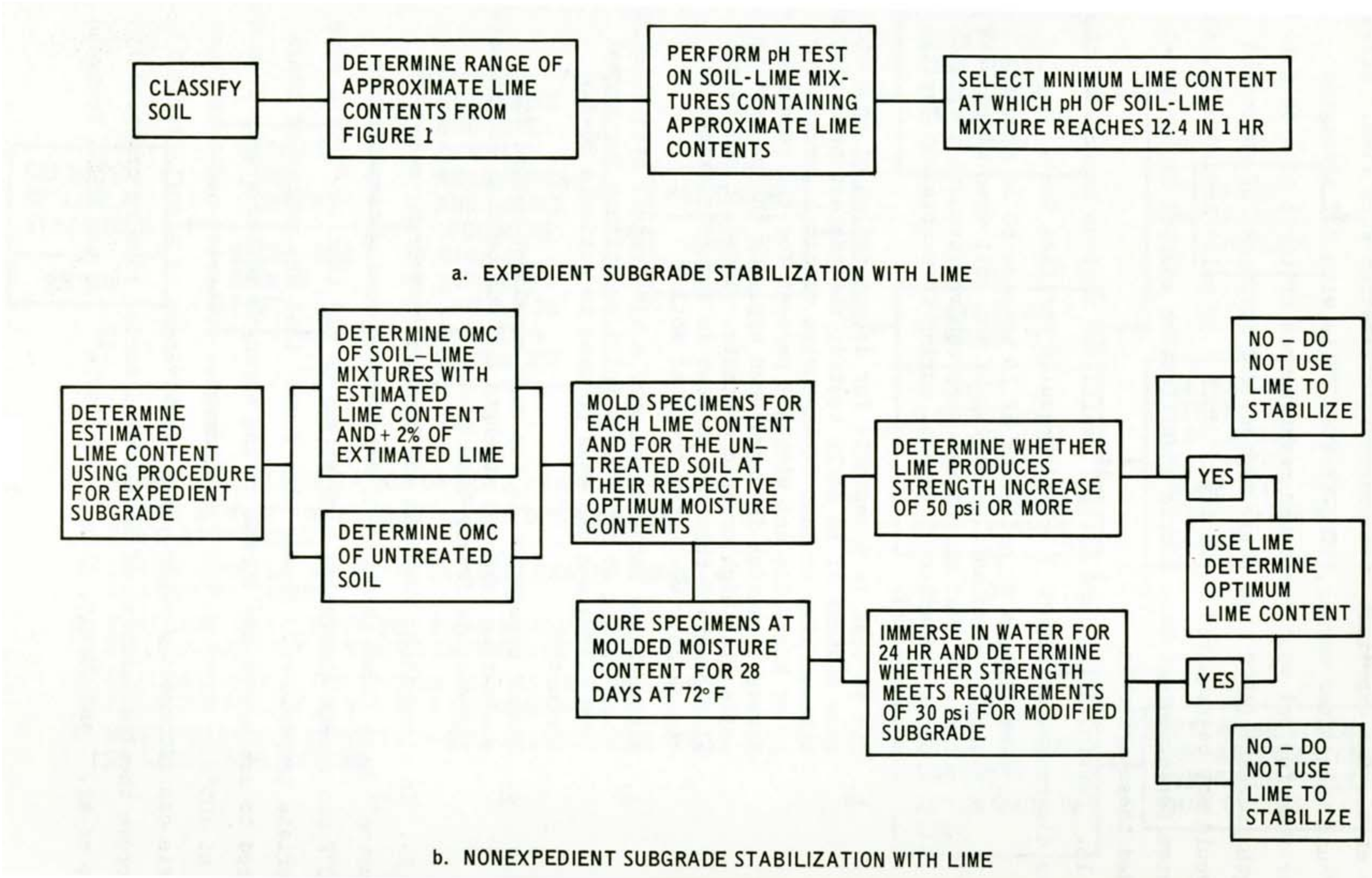


Figure 6.2 Expedient and Non-Expedient Subgrade Stabilization with Lime (from Dunlap,1975)

Validations and/or modifications to improve this subsystem have been investigated by Dunlap et al.(1971) and Currin, Allen, and Little (1976). Specifically the improvements have centered on using accelerated curing procedures to estimate 28-day strengths. Originally, Thompson (1970) recommended curing lime-treated specimens for 48 hr at 120° F to estimate 28-day strengths. However, Townsend and Donaghe (1979) found that accelerated curing at 120° F could create abnormally high strengths and recommend accelerated curing of 65 hrs. at 105° F.

Philosophy of Mix Design System for Restoration Using Lime

Townsend and Donaghe (1979) developed a mix design system for the restoration of levee landslides that is applicable for most soil-lime mixtures. Figure 6.3 presents the flow diagram for determining lime contents and verifying lime treatment susceptibility of clays. The system is predicated on two situations: (a) lime treatment is to modify soil and decrease potential cracking caused by alternating shrink—swell cycles; that is no major strength improvement is required; and (b) lime treatment is to increase strength or durability, in which case major strength improvements are required. The system uses TM *5—887—5* criteria; i.e., (P1 > 10) to verify that lime should even be considered for soil in question. The pH test is subsequently used to estimate approximate design lime *modification* content. Since the pH test does not necessarily verify that a soil will be modified or stabilized if the pH—lime percentage is added to it, modification and strength tests must be conducted.

If modification only (situation (a)) is desired, a P1 reduction of 50 percent is considered as a reasonable demonstration that increased workability and reduced plasticity will result with lime treatment. A reduction in P1 to less than 15 percent or classification of the lime—treated soil as a silt (MH or ML) will probably preclude cracking. If lime treatment fails to provide these basic benefits, the soil is judged nonsusceptible. Should the situation arise that the amount of strength improvement due to lime modification is desired, optional strength tests as shown can be performed. Compaction of test specimens should be comparable to that anticipated in the field.

It is envisioned that 1.4-in.-diam U/C specimens can be prepared from the 4-in.diam. compaction specimens, in which case a suite of U/C tests can be performed to provide water content and density effects on strength. Alternatively, just the compaction specimen closest to optimum conditions (based upon wet densities, although dry is preferable), can be used to prepare U/C specimens. A soil-lime strength increase of 100 percent of the raw soil strength is deemed as acceptable for demonstrating a positive benefit by adding lime; if less than 100 percent, lime is judged of little benefit. This strength increase can be verified by accelerated curing if expedient solutions are required, although normal 28-day curing at room temperatures is preferred.

If strength and durability (situation (b)) are desired, a U/C strength increase of 3.6 tsf *(50* psi) at optimum conditions and at standard compaction effort due to lime treatment and a durability of 2.16 tsf (30 psi) after 24 hrs. of soaking are considered as a reasonable demonstration of lime benefits. These strengths probably are more than adequate, but experience indicates that a 3.6 tsf (50 psi) U/C-strength increase provides a satisfactory performance. U/C strengths of

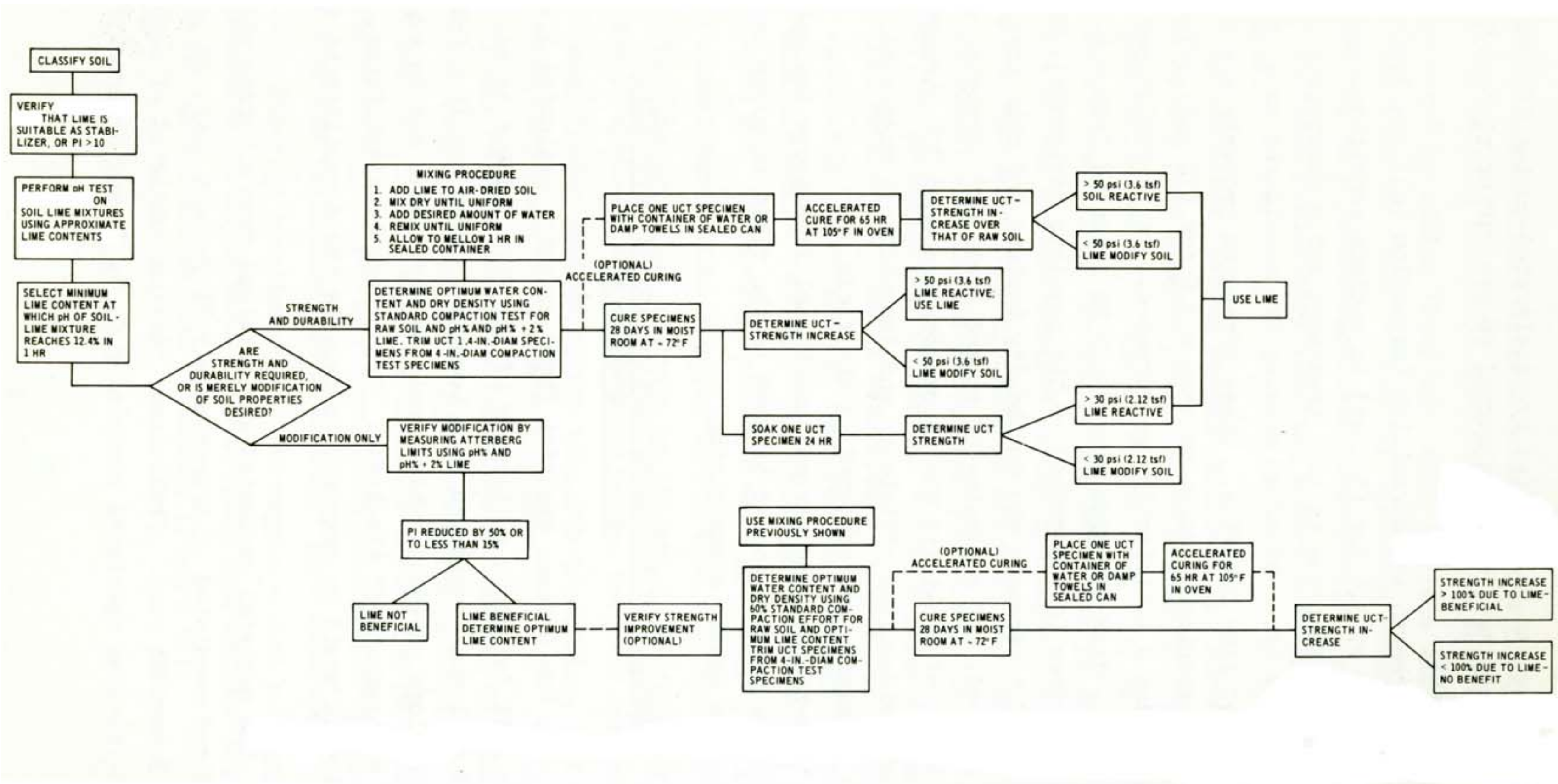


Figure 6.3 Flow Diagram for Lime Stabilization (from Townsend and Donaghe, 1979)

lime treated specimens greater than 100 psi. usually meet the criteria of a 50-psi strength increase, and can be used to eliminate testing untreated specimens (Biswass, 1972). It is envisioned that two or three l.4-in.-diam UCT specimens would be prepared from a 4-in.-diam compaction specimen (as described in the previous paragraph), which would be used for strength and durability testing and even accelerated curing if desired. If estimates of soaked field U/C strengths are needed, a value of one-third the laboratory unsoaked UCT strength to account for field-mixing efficiency and immersion is suggested, provided comparable densities are achieved in the field.

One-half to one percent additional lime above the optimum laboratory lime content should be used in the field to cover construction losses and uneven distribution.

LIME STABILIZATION BENEFITS

Plasticity Reduction

Adding lime at the *modification* optimum percentage or higher, causes an immediate reduction in plasticity and increased granulation of the soil. These actions result in increased workability and the establishment of a working platform. The plasticity reduction is mainly due to an increase in plastic limit (PL), accompanied by a slight decrease in liquid limit (LL), as illustrated in Figure 6.4

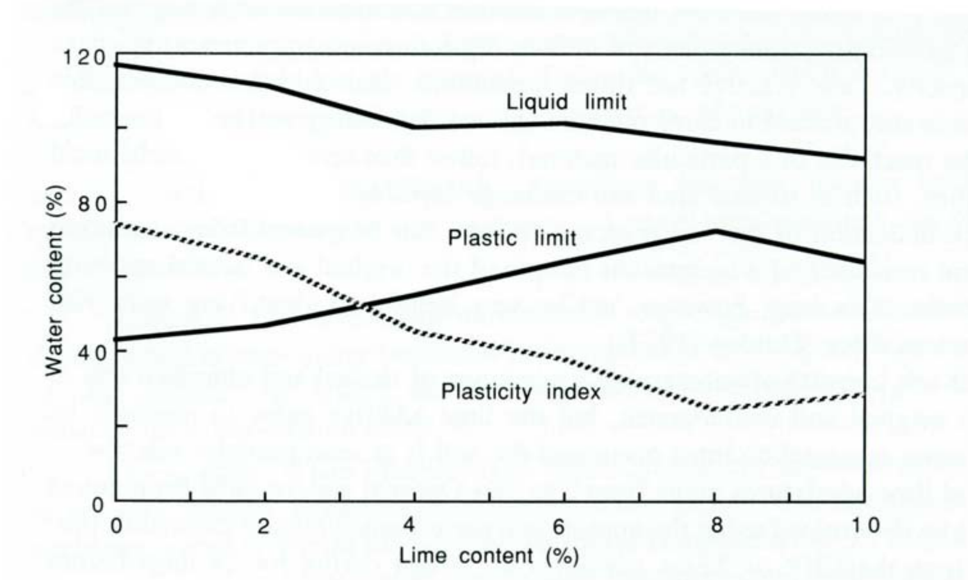


Figure 6.4 Effect of Lime Content on Soil Plasticity (from Hausmann, 1990)

Compaction and Strength Properties of Soil –Lime Mixtures (Hicks, (2002); Townsend, 1979)

The addition of lime to soil causes a plasticity reduction and "granulation" of the soil particles due to cation exchange and flocculation. As a consequence, the more granular soil causes the optimum water content to increase and maximum dry density to decrease; i.e., the moisture – density curves shifts down and to the right. Figure 6.5 illustrates this effect This effect is further increased by delaying compaction once the lime is added. This granulation effect on moisture –density must be considered in mix designs, and field compaction specifications.

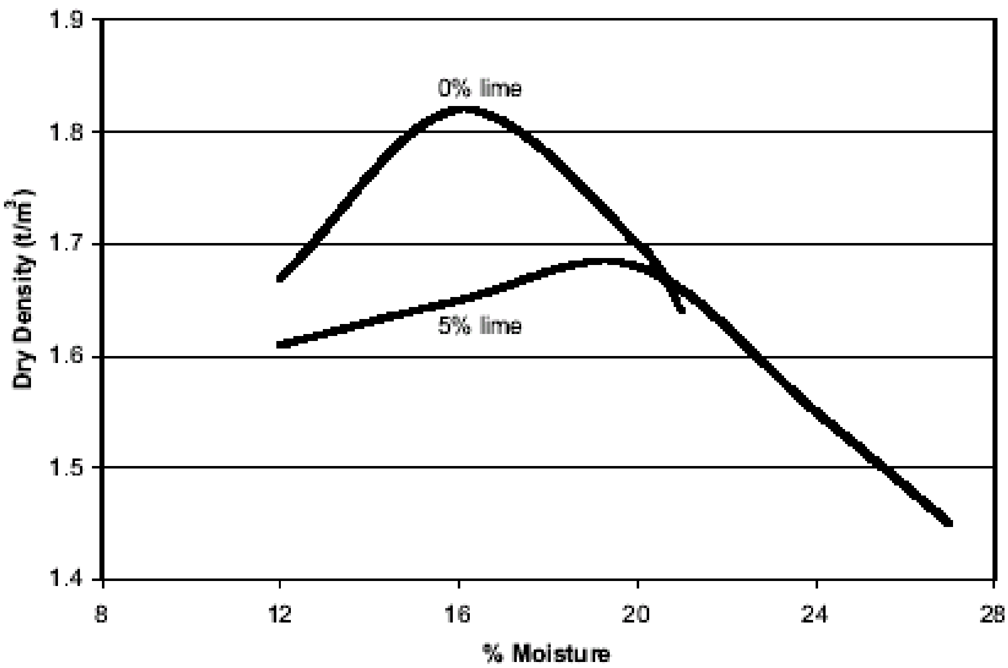


Figure 6.5 Effect of Lime Content on Moisture – Density Relationship (from Hicks, 2002)

Much as a *modification* optimum lime content exists, a *stabilization* optimum lime content exists. High lime contents will not necessarily produce high early strengths. Figure 6.6 illustrates the variations in strength with time and lime content for lime-stabilized materials. Figure 6.6 shows for this soil, the *stabilization* optimum lime content is approximately 6 to 7 percent for 7-day or 21- day curing.

ANTICIPATED PROPERTIES OF COMPACTED SOIL – LIME MIXTURES

Thompson (1970a) provides the following correlations for U/C compression specimens (q_u = unconfined compressive strength):

Split tensile strength, (psi)	$0.13\ q_u$
Cohesion, (psi)	$9 + 0.29\ q_u$
ϕ	25 – 35°
E_c (ksi) @ σ_3=15psi	$10 + 0.124\ q_u$
Poisson's ratio μ	0.1

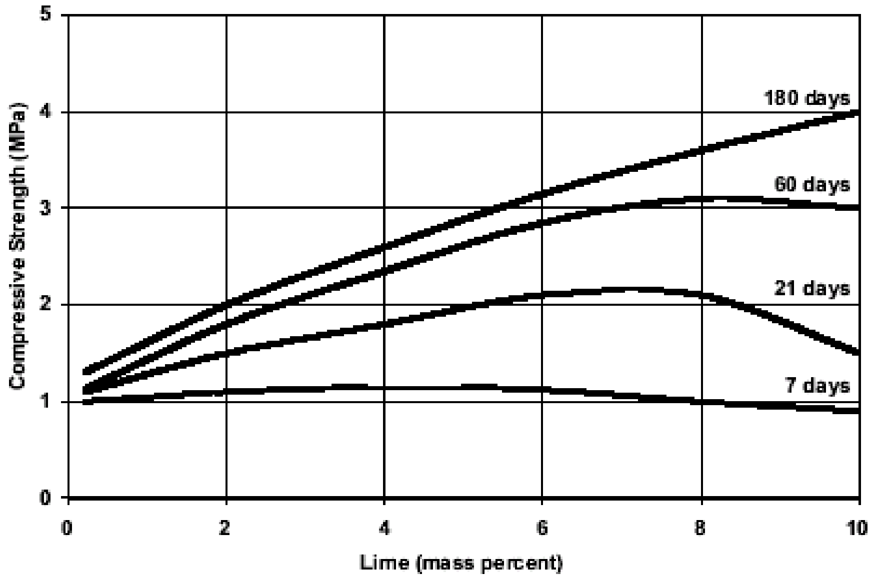


Figure 6.6 Effect of Time and Lime Content on Compressive Strength (Hicks, 2002)

EFFECT OF COMPACTION DELAY ON LIME-TREATED CLAYS

One problem encountered during construction will concern the effect of delay in compaction after mixing lime with the soil. Accordingly, Figure 6.7 summarizes the effects of compaction delay on strength and density. These data show that the longer the mellowing time, the lower the density and corresponding strength. The greatest decreases occur during the initial 24 hrs., with perceptible but insignificant changes occurring with longer mellowing times. The levee clay from DeGonia exhibited a 30 percent strength decrease over 24 hr, while the Roundaway Bayou clay lost about *75* percent due to delaying compaction 24 hrs. According to Howard and Bara (1976), the maximum strength and density reductions occur during the initial 8 hr of mellowing time. These reductions in density and corresponding reduction in strength are attributed to granulation of the loose soil particles by weak cementation as the soil mellows. However, if specimens were compacted to the same density, approximately equal strengths could be obtained for specimens with different mellowing times, up to 72 hrs. Hence delay in field mixing and compaction is not detrimental except for the additional costs to provide extra compaction to achieve comparable higher densities where delay was minimal. (Townsend, 1979)

IMMERSION EFFECTS ON DURABILITY

Figure 6.8 illustrates the effects of immersion on the U/C strength of lime treated soils. As shown, lime treatment at the *modification* percentage dramatically improved the soaked strength. The untreated soils upon soaking were very weak or completely slaked. Whereas the lime treated soils a strength loss upon soaking of 14 – 50%. Thompson (1970) suggests only slightly detrimental immersion strength losses of 15 – 30 %.

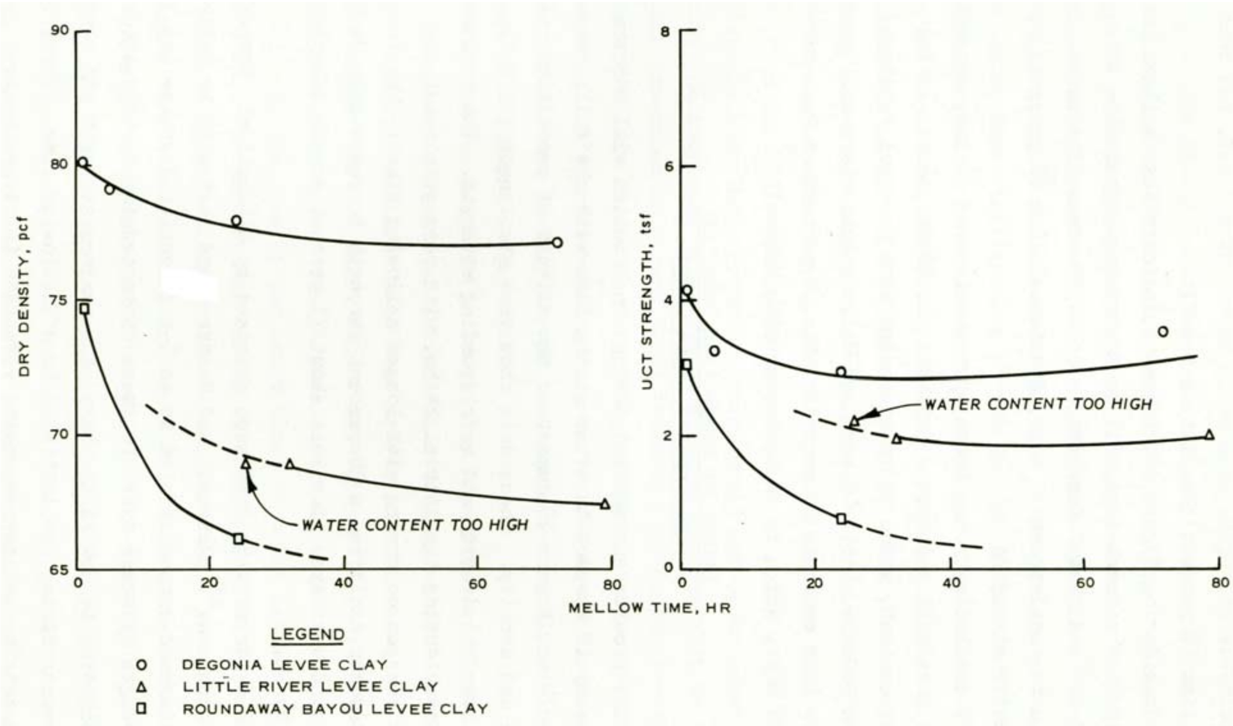


Figure 6.7 Effect of Mellow Time on Density and Strength (Townsend, 1979)

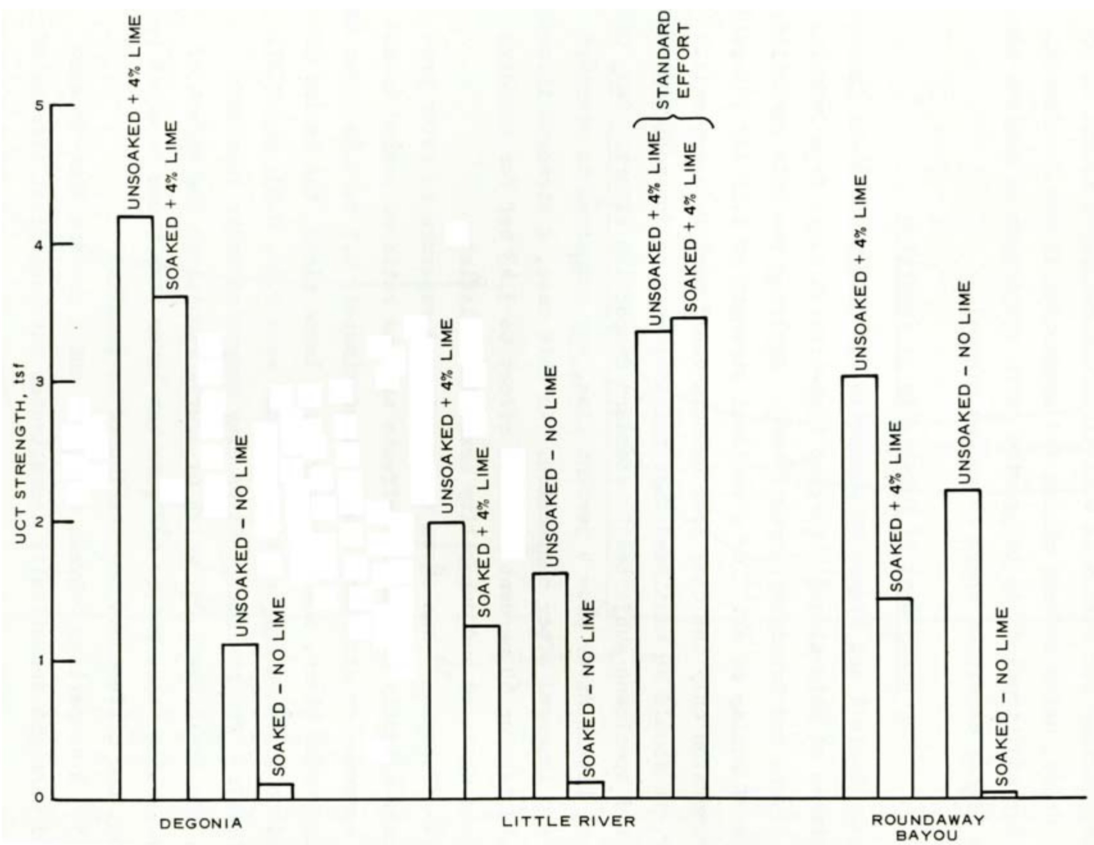


Figure 6.8 Effect of Immersion on Strength of Lime -Treated and Natural Soil
(Townsend, 1979)

CONSTRUCTION CONSIDERATIONS

FDOT Standard Specifications for Road and Bridge Construction (2000) section 165 addresses construction considerations for lime treated subgrades.

http://www11.myflorida.com/specificationsoffice/y2kBook/toc.htm

Lime Spreading - Prior to mixing either from a bulk source or by bag, dry lime can be applied to the soil at a specified percent. Bagged lime is generally the simplest, but most expensive method, due to greater labor costs and slower operations caused by increased handling. Nevertheless, for small projects, large equipment may not be warranted, and bagged lime may be the only practical method. Generally, the bags are spaced to provide the desired application percentage for the lift, slit, and the lime dumped into piles or windrows across the lift. Spreading is then accomplished by hand raking, or a spiked-tooth harrow, or drag-pulled by tractor or truck. Immediately, thereafter, the lime is sprinkled to reduce dusting. Several innovative methods with varying degrees of success have been used to spread bagged lime. A fertilizer spreader has been used unsuccessfully as too many passes were required to spread the desired amount of lime.

Alternatively, a procedure can be used whereby the bagged lime is emptied into a front-end loader bucket and then spread by the bucket. The Panama Canal Company reportedly used a method of spreading bagged lime by spacing the bags, slitting them, and exploding detonating primer cord that had been wrapped around the bags.

For large projects, where dusting is not a major problem, spreading bulk lime from large (15- to 24-ton) self-unloading transport trucks is common practice. The newer units are pneumatically operated, and the lime is blown from the tanker compartments to a cyclone spreader or pipe spreader bar mounted at the rear. For exceptionally large projects, central batching plants have been used. Self-unloading tank trucks are probably the least expensive and most practical method of spreading lime, providing the size of the project warrants their use.

If dry quicklime is being applied, precautions must be taken to minimize the danger of chemical burns. Some progress has been made to minimize this danger by using pelletized or granular quicklime. The advantages of quicklime are lower cost, more lime per ton (approximately 25 percent), and faster reaction times. However, justification, capitalizing on these advantages, must outweigh the hazards involved and protective equipment required.

Slurry Application. Although dry lime application has been the most widely used procedure, the slurry method due to ease, reduced dust problems, and better distribution is becoming more popular. However, some consideration should be given to the effects of slurry treatment and water content desired for compaction. A typical slurry mix that has been used is 1 ton of lime per 500 gal of water, which yields approximately 600 gal of slurry* containing 31 per-cent lime solids. For control purposes the specific gravity of the slurry can be checked; for example, for the preceding mix proportion the specific gravity should be approximately 1.18 to 1.23.

An alternative method of slurry production, which eliminates batching trucks, involves a compact jet slurry mixer. In this device, water at 70-psi pressure and hydrated lime are charged continuously in a jet mixing bowl, where slurry is produced instantaneously.(Lime Assoc, 1972)

Double Application of Lime. In cases of extremely plastic clays (P1 > *50)*, it may be advantageous to add the lime in two applications. An application of 2-3 percent is added first, partially mixed, and allowed to mellow from several days to a week. This mellowing period is to modify the soil by reducing plasticity and increasing workability, so that final pulverization and the second application of the remaining lime can be accomplished more easily for stabilizing the soil.

Mixing and Watering

A key factor in obtaining satisfactory soil-lime treatments is providing adequate pulverization and intimate mixing of the soil and lime. Typically lift thicknesses of 6-9 in. are pulverized and mixed. As stated previously, double lime application may be required in highly plastic clays (PI> *50)* to achieve satisfactory pulverization. Although field clays may be at water contents well above optimum, it may be necessary to sprinkle the limed soil liberally to achieve good distribution of the lime. Ideally after mixing and sprinkling, the soil-lime mixture would be at its optimum water content just prior to compaction. However, realistically, considering most environmental conditions, drying to optimum water content may be impractical. In these wet of optimum situations, densities and corresponding strengths and durabilities may be reduced, as explained earlier. Although disk harrows and grader scarifiers are suitable for preliminary mixing, high-speed rotary pulverizers are highly recommended and should be used unless the project size warrants other means. Most likely for heavy clays, blade mixing will probably be unsatisfactory except for only minor preliminary mixing.

Pulverization and mixing requirements generally specify that soil-lime mixtures should be pulverized so that 100 percent passes the 1-in, sieve and 60 percent passes the No. 4 exclusive of nonslaking fractions. However, the South Dakota Highway Department only requires that *95* percent, based on wet weight of pulverized soil, passes the 1-1/2-in, sieve.

Compaction and Finishing

For maximum development of strength and durability, soil-lime mixtures should be compacted to high densities, i.e., *95* percent standard. However, in some cases of landslide restoration only semi-compaction is used, i.e., four to six passes of a crawler tractor or by routing of hauling equipment. Nevertheless under these conditions Townsend (1979) reported that adequate modification of clays and significant strength gains can be achieved using semi-compaction.

Depending upon construction sequence, breakdowns, and weather, delays between mixing the lime with the soil, placement of the soil lime mixture, and compaction may arise. general guidance suggests immediate compaction whenever possible, but delays up to 4 days are not detrimental for fine-grained soils. Figure 6.7 suggests that compaction delays of 24 hrs. can produce strength decreases of 30-75 percent over 1-hr compaction delays. The Bureau of Reclamation (Howard and Bara, 1976) has shown that the initial 8 hrs. between adding the lime and compaction are the most critical for obtaining highest strengths, and delays between 8 and 12 hr after mixing resulted in essentially similar strengths. However, this strength loss with compaction delay can be overcome by compacting to higher densities. When long delays, i.e., 2 weeks or more are unavoidable, consideration should be given to a second application of a small amount of additional lime.

Lime Treatment of Expansive Soils

Lime treatment is the most widely used and effective technique to minimize the detrimental effects of expansive soils. Table 6.1 provides guidance concerning swelling potential. (Holtz, 1969). Typically, lime contents at the *modification* (pH) percentage are used.

Table 6.1 Correlations of Swelling Potential vs. Atterberg Limits (Holtz, 1969)

Plasticity Index	Liquid Limit	Swelling Potential
< 18	<39	Low
15 – 28	39 – 50	Medium
25 – 41	50 – 63	High
> 35	> 63	Very High

Hayward Baker uses a lime slurry pressure injection system in which lime is injected to depths up to 40 ft. at pressures ranging from 30 to 200 psi on spacings of 2 to 5 feet. Pressure injection is best suited for fissured soils whereby the lime slurry is injected through the fissures to seal and with time stabilize the expansive soil. Figure 6.9 illustrates the lance-type injection system.

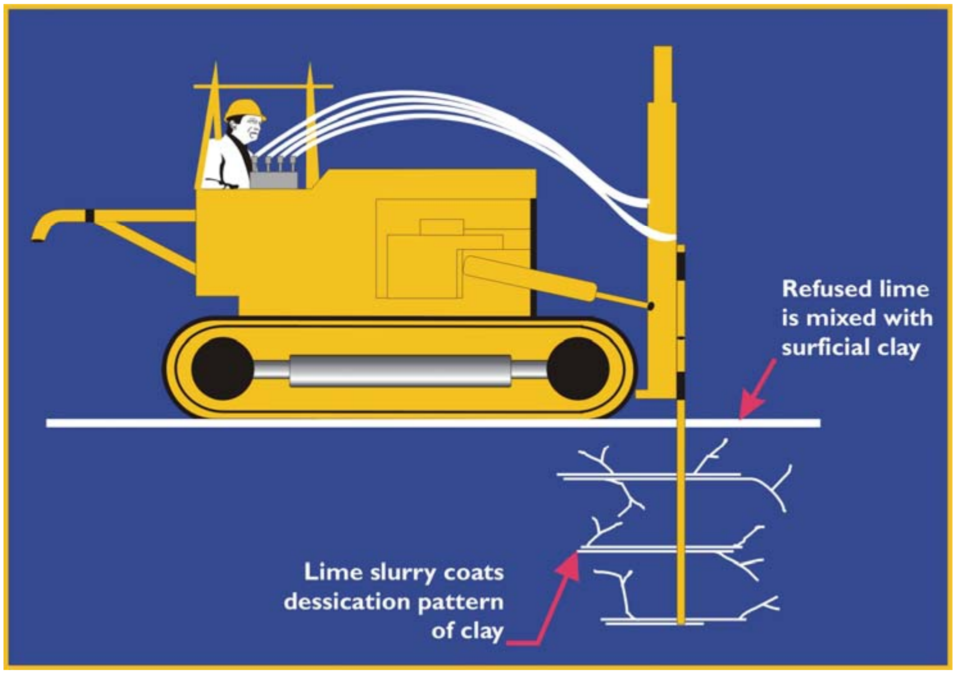


Figure 6.9 Lime Slurry Pressure Injection System (from Hayward Baker)

Soil–Cement Stabilization – Mix Design

Inasmuch as soil – lime stabilization is favorable for clayey fine-grained cohesive soils, soil – cement stabilization is favorable for coarse-grained cohesionless soils (see Figure 6.1). A mix design procedure for soil – cement is: (TM 5-822)

Step 1: Use Table for estimating cement contents

Soil Classification	Initial Estimated Cement Content percent dry weight
GW, SW	5
GP, GW-GC, GW-GM, SW-SC, SW-SM	6
GC, GM, GP-GC, GP-GM, GM-GC, SC, SM, SP-SC, SP-SM, SM-SC, SP	7
CL, ML, MH	9
CH	11

Step 2: Using the estimated cement content, conduct moisture-density tests to determine the maximum dry density and optimum water content of the soil-cement mixture.

Step 3: Prepare triplicate samples of the soil-cement mixture for unconfined compression and durability tests at the cement content selected in step 2 and at cement contents 2 percent above and 2 percent below that determined in step 2. The samples should be prepared at the density and water content to be expected in field construction. For example, if the design density is 95 percent of the laboratory maximum density, the samples should also be prepared at 95 percent. Cure the specimens for 7 days in a humid room before testing. Test three specimens using the unconfined compression test in accordance with ASTM D 1633, and subject three specimens to durability tests, either wet-dry (ASTM D 559) or freeze-thaw (ASTM D 560) tests as appropriate.

Step 4: Compare the results of the unconfined compressive strength and durability tests with the requirements shown in Tables 6-2 and 6-3. The lowest cement content which meets the required unconfined compressive strength requirement and demonstrates the required durability is the design cement content. If the mixture should meet the durability requirements but not the strength requirements, the mixture is considered to be a modified soil. If the results of the specimens tested do not meet both the strength and durability requirements, then a higher cement content may be selected and steps 1 through 4 above repeated.

Table 6.2 Minimum Unconfined Compressive Strength for Cement, Lime, Lime-Cement, and Lime-Cement-Fly Ash Stabilized Soils

	Minimum Unconfined Compressive Strength, psf*	
Stabilized Soil Layer	**Flexible Pavement**	**Rigid Pavement**
Base course	750	500
Subbase course, select material or subgrade	250	200

* Unconfined compressive strength determined at 7 days for cement stabilization and 28 days for lime, lime fly ash, or lime cement-fly ash stabilization.

Table 6.3 Durability Requirements

Type of Soil Stabilized	Maximum Allowable Weight Loss After 12 Wet-Dry or Freeze-Thaw Cycles Percent of Initial Specimen Weight
Granular, PI < 10	11
Granular, PI > 210	8
Silt	8
Clays	6

Construction considerations for cement treated subgrades are presented in section 170 of FDOT Standard Specifications for Road and Bridge Construction (2000).

Lime – Fly Ash – Mix Design

Obviously, lime-fly ash (LF) stabilization is intended for those soils lacking sufficient natural silica or alumina to react with the lime. In this case, the fly ash is the source of silica and/ or alumina for pozzonalic reactions with the lime. In this context, nonplastic soils; i.e., sands, and gravels, are more suitable for lime-fly ash stabilization than cohesive clays. INDOT (2003) recommends lime-fly ash suitable soils are those containing less than 10% passing No. 200, and the plasticity index range is 10 < PI < 20.

Mix design with LF is different than stabilization with lime of cement, in that three (3) components; lime, fly ash, and aggregate, are to be combined. Consequently, the lime: fly ash ratio, and percentage of lime- fly ash must be determined. Typically, 3 to 10 percent lime and 15 to 30 percent total LF mixture are added. LF ratios ranging from 1:2, to 1:4 are used. Alternatively, by considering the fly ash as fines, a lime: fines ratio ranging from 1:4 to 1:7 could be used. (Townsend, 1976). The following mix design procedure could be used: (TM 5-822)

1. Evaluation of lime – fly ash. This step explores the compatibility of the lime + fly ash proposed for stabilization using ASTM C 593-95. ASTM C 593 uses Proctor size specimens of a mix composed of 180 g lime + 360 g. FA + 1480 g sand, compacted

under modified conditions and cured 7 days at 100° F. Compressive strengths of these specimens must exceed 400 psi.

2. Determination of optimum fines content. This step develops the percentage of fly ash + soil fines need to develop the maximum density. The premise is that strength and durability are directly related to the density. Basically, higher strengths and durability are obtained when the fine matrix material is able to "float" the coarser aggregate particles. The quantity of fines matrix required for the maximum dry density of the total mixture is the optimum fines content. This is accomplished by performing moisture – density tests using fly ash contents ranging from 10 to 30 percent of the total mixture. The design fly ash content is 2 % above that providing the greatest density. Alternatively, single point compaction tests could be made at fly ash contents of 10-30 percent, and plotting density vs. fly ash content to determine the optimum fly ash content.

3. Determination of Strength and Durability. This step prepares modified Proctor specimens at LF ratios of 1:2 to 1:5 at fly ash contents determined from the proceeding step. Using criteria from Table 6-2 and 6-3 the minimum LF mixture satisfying the criteria is selected. If criteria is not met, the addition of 1–2 percent cement could be considered and a LCF mixture evaluated.

Asphalt – Soil Mix Design

Asphalt stabilization of soils is usually intended to provide some cohesion to nonplastic soils for strength improvement, and to make a cohesive soil less susceptible to strength loss due to increased moisture; i.e., "waterproofing." Due to mixing problems, asphalt stabilization is better for granular soils; although pretreatment of clays with lime prior to adding asphalt is viable.

Mix design criteria for design of bituminous stabilized soils and aggregates are based almost entirely on stability and gradation requirements.

Hicks (2002) provides Figure 6.10 as guidance.

Types of bitumen. (TM 5-822) Bituminous stabilization is generally accomplished using asphalt cement, cutback asphalt, or asphalt emulsions. The type of bitumen to be used depends upon the type of soil to be stabilized, method of construction, and weather conditions As a general rule, the best results are obtained when the most viscous liquid asphalt that can be readily mixed into the soil is used. For higher quality mixes in which a central plant is used, viscosity-grade asphalt cements should be used. Much bituminous stabilization is performed in place with the bitumen being applied directly on the soil or soil aggregate system and the mixing and compaction operations being conducted immediately thereafter. For this type of construction, liquid asphalts, i.e., cutbacks and emulsions, are used. Emulsions are preferred over cutbacks because of energy constraints and pollution control efforts. The specific type and grade of bitumen will depend on the characteristics of the aggregate, the type of construction equipment, and climatic conditions. Generally, Table 6.4 illustrates the types of bituminous materials used for various soil gradations.

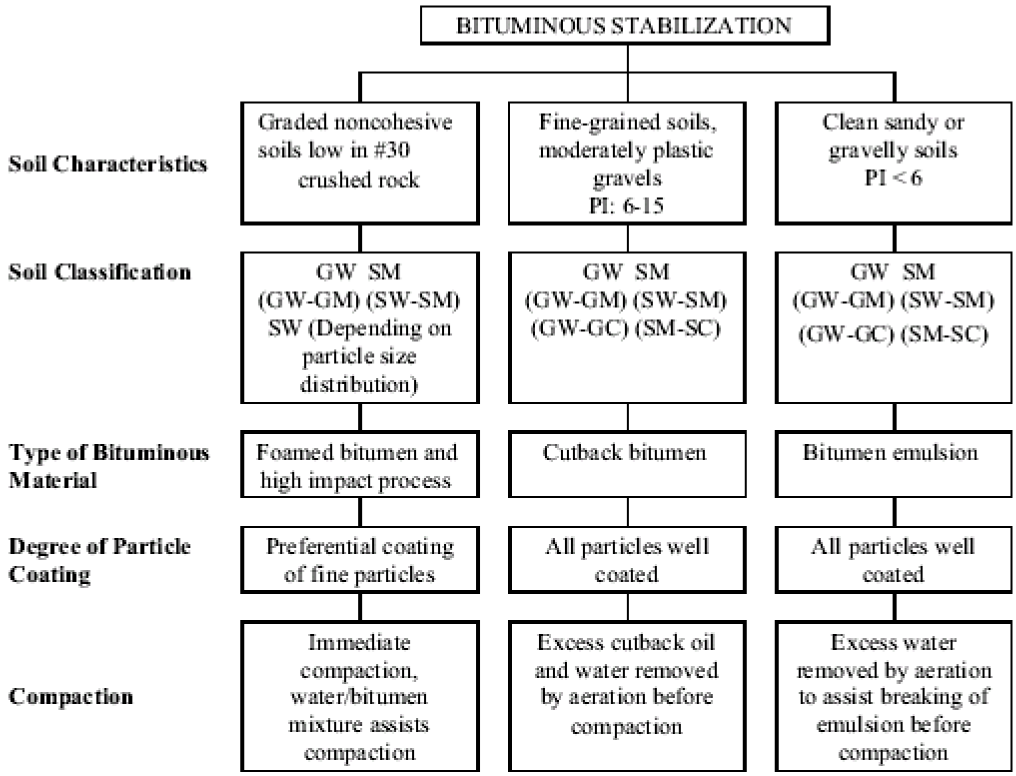


Figure 6.10 Guide for Selecting Asphalt – Soil Compatibility (from Hicks, 2002)

Table 6.4 Suggested Bituminous Types vs. Soil Gradation (from TM 5-822)

Soil Type		
Open graded aggregate	Well-graded aggregate with little or no fines	Aggregate with a considerable percentage of fine aggregate and fines.
Rapid- and medium-curing liquid asphalts: RC-250, RC-800, MC-3000.	Rapid and medium-curing liquid asphalts RC-250, RC-800, MC-250, and MC-800	Medium-curing liquid asphalt MC-250, MC-800.
Medium-setting asphalt emulsion MS-2 and CMS-2.	Slow-curing liquid asphalts SC-250 and SC-800.	Slow-curing liquid asphalts SC-250 and SC-800
	Slow-curing liquid asphalts SC-250 and SC-800.	Slow-setting asphalt emulsions SS-1, SS-01h, CSS-1, and CSS-lh.

The simplest type of bituminous stabilization is the application of liquid asphalt to the surface of an unbound aggregate road. For this type of operation, the slow- and medium-curing liquid asphalts SC-70, SC-250, MC-70, and MC-250 are used.

For subgrade stabilization, the following equation may be used for estimating the preliminary quantity of cutback asphalt to be selected:

$$p = \frac{0.02(a) + 0.07(b) + 0.15(c) + 0.20(d)}{(100 - S)} \times 100$$

Where:

p = percent cutback asphalt by weight of dry aggregate

a = percent of mineral aggregate retained on No. 50 sieve

b = percent of mineral aggregate passing No. 50 sieve and retained on No. 100 sieve

c = percent of mineral aggregate passing No. 100 and retained on No. 200 sieve

d = percent of mineral aggregate passing No. 200

S = percent solvent

The final design content of cutback or emulsified asphalt should be selected based upon the results of the Marshal Stability test procedure. The minimum Marshall Stability recommended for subgrades is 500 pounds. If a soil does not show increased stability when reasonable amounts of bituminous materials are added, the gradation of the soil should be modified or another type of bituminous material used.

REFERENCES

National Lime Association publications: http:// www.lime.org/publications.html

Biswass, B., (1972) Study of Accelerated Curing and Other Factors Influencing Soil Stabilization, Ph. D. Thesis, Texas A&M University.

Currin, D.D., Allen, J.J., and Little, D.N. (1976) "Validation of Soil Stabilization Index System with Manual Development", Tech. Dept. SRL-TR-76-0006, Frank J. Seiler Res. Lab.

Dunlap, W.A., Epps, J.A., and Gallaway, B.M., "United States Air Force Soil Stabilization—Index System - A Validation," AFWL-TR-73—150, Jan 1975.

Eades, J. (1962), Reaction of $Ca(OH)_2$ with Clay Minerals in Soil Stabilization, PhD Dissertation Univ of Ill.

Eades, J.L. and Grim, R.B., "A Quick Test to Determine Lime Requirements for Lime Stabilization," Highway Research Record No. 3, 1966, National Academy of Sciences, National Research Council, Highway Research Board, Washington, D. C.

Epps, J.A., Dunlap, W.A., and Gallaway, B.M., "Basis for the Development of a Soil Stabilization Index System," Technical Report No. AFWL-TR-70-176, Dec. 1971, Air Force Weapons Laboratory, Kirtland Air Force Base, N. Mex.

FDOT Specifications for Road and Bridge Construction (2002) http://www11.myflorida.com/specificationsoffice/y2kBook/toc.htm

Hicks, G.R. (2002) Alaska Soil Stabilization Design Guide Design Guide FHWA-AK-RD-01-6B Alaska DOT Fairbanks, AK 99709-5399 http://www.dot.state.ak.us/stwddes/research/assets/pdf/fhwa_ak_rd_01_06b.pdf

Ho,C. and Handy, R.L. (1963) Electrokinetic Properties of Lime Treated Bentonites" Proc. 12th Nat. Conf. On Clays and Clay Minerals, Pergamon Press p.267

Holtz, W.G. (1969) Volume Change in Expansive Clay Soils and Control by Lime Treatment" Proc. 2nd Int. Research and Engineering Conf. On Expansive Clay Soils, p. 157-173, Texas A& M Press College Station, TX.

Howard, A.K. and Bara, J.P., "Lime Stabilization on Friant-Kern Canal," Dec 1976, Report REC-ERC-76—2O, Bureau of Reclamation Engineering and Research Center.

Indiana DOT (2002) Design Procedures for Soil Modification or Stabilization, Materials and Tests Division - Geotechnical Section, Indianapolis, ID 46219 http://www.in.gov/dot/business/design/geotech/smod.pdf

Ingles, 0.0. and Metcalf, J.B., Soil Stabilization: Principles and Practice, Wiley, New York, 1973.

National Lime Association (1972) Lime Stabilization Construction Manual, 5th Ed. Bull. 326

"State of the Art: Lime Stabilization, Reactions, Properties, Design, Construction," Transportation Research Circular 180, Sep 1976, Transportation Research Board, Washington, D. C.

Thompson, M. F., (1970) "Suggested Method for Mixture Design Procedure for Lime—Treated Soils," Special Procedures for Testing Soil and Rock for Engineering Purposes, 5th ed., American Society for Testing and Materials, Philadelphia, Pa., STP –479, pp 430-440.

Thompson, M. R. (1964) Lime Reactivity of Illinois Soils as it Relates to Compressive Strength, PhD Dissertation, Univ. of Ill. Urbana, Ill.

Thompson, M.R. and Eades, J.L., (1970a)"Evaluation of Quick Test for Lime Stabilization," Journal, Soil Mechanics and Foundations Division, American Society of Civil Engineers, Vol. 96, No. 5M2, pp 795-800.

Thompson, M.R. and Robnett, Q.L., "Pressure Injection Lime Treatment of Swelling Soils," TRB Record 68, 1976.

TM *5—887—5,* Office, Chief of Engineers, Department of the Army, "Soil Stabilization Emergency Construction," Technical Manual, 25 May 1966, Washington, D. C.

Townsend, F.C. and Donaghe, R.T., "Investigation of Accelerated Curing of Soil-Lime and Lime-Flyash—Aggregate Mixtures," Technical Report *5-76-6*, Aug 1976, U.S. Army Engineer Waterways Experiment Station, CE, Vicksburg, Miss.

(TTI Report 1994-5, "Reduction of Sulfate Swell in Expansive Clay Subgrades in the Dallas District," S. Bredenkamp and R. Lytton. 1994.)

US Army (1994) TM 5-822-14 Soil Stabilization For Pavements

Wright, B.J., "Lime Slurry Pressure Injection Tames Expansive Clays," Civil Engineering, American Society for Civil Engineers, Oct 1913.

Appendix to Chapter 6 – Eades and Grim pH Test (ASTM D 6276)

Procedure: To determine the amount of lime to be added to soils to produce a pH of 12.4 or equal to a pH of lime itself. The optimum lime content shall be determined corresponding to the maximum pH of lime-soil mixture. (See Figure A-1).

- Representative samples of air-dried, minus No. 40 soil equal 20 gm of oven-dried soil are weighed to the nearest 0.1 gm and poured into 150-ml (or larger) plastic bottles with screw tops.
- It is advisable to set up five bottles with lime percentages of 3, 4, 5, 6, 7. This will insure, in most cases, that the percentage of lime required can be determined in one hour. Weigh the lime to the nearest 0.01 gm and add it to the soil. Shake to mix soil and dry lime.
- Add 100 ml of CO2-free distilled water to the bottles.
- Shake the soil-lime and water until there is no evidence of dry material on the bottom. Shake for a minimum of 30 seconds.
- Shake the bottles for 30 seconds every 10 minutes.
- After one hour, transfer part of the slurry to a plastic beaker and measure the pH.
- Record the pH for each of the lime-soil mixtures. If the pH readings go to 12.40, the lowest percent lime that gives a pH of 12.40 is the percent required to stabilize the soil. If the pH did not go beyond 12.30 and 2 percent lime gives the same reading, the lowest percent that gives a pH of 12.30 is that required to stabilize the soil. If the highest pH is 12.30 and only 1 percent lime gives a pH of 12.30, additional test bottles should be started with larger percentages of lime.

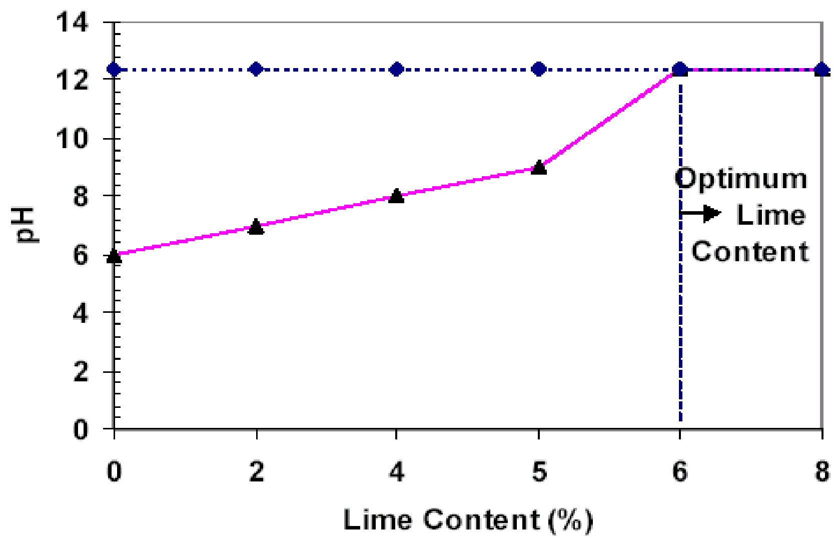


Figure A-1. Example of pH Lime Test (from Indiana DOT, 2002)

Chapter 7

GROUTING

INTRODUCTION

Grouting is defined as the injection of pumpable materials into soil or rock, under pressure through boreholes. The intent being to; fill the voids in the ground, strengthen the soil, stabilize loose deposits, and create a less permeable medium.

Grouting categories are classified according to the method of injecting the grout into the ground. However, other classifications could depend upon the type of grout material. Figure 7.1 presents the basic grouting classes, and their historical use in the USA is:

Slurry Grouting (intrusion) – 1890’s

Penetration (permeation) Chemical Grouting – 1950’s

Displacement Grouting/Compaction (“slab-jacking”) Grouting – 1950’s

Jet Grouting – 1980’s

Fracture Grouting – 1990’s

Slurry Grouting – Slurry grouting consists of pressure injecting flowable suspensions of cement /clay grouts into open crack, fissures, and voids. Early uses were primarily to seal dam foundations via cut-off walls against leakage.

Penetration Grouting – Penetration grouting is a process by which the pore spaces in soil or the joints in rock are filled with grout, so as not to fracture or create volume change in the soil formation. Consequently, injection pressures are usually limited to less than 20 kPa per meter of depth (1 psi/ft). Either particulate or chemical grouts are used and the soils must be fairly permeable; i.e., coarse grained to allow passage of the grout. Typically, grouting materials are cement, cement-bentonite mixtures, or clays are used in medium to coarse sands. Whereas chemical grouts are used for fine sands.

Displacement (Compaction) Grouting – Compaction grouting is the opposite of penetration grouting, where rather than the grout penetrating into the soil voids, the thick grout displaces the soil. Compaction grout consists of a low slump concrete mortar injected into soft or loose soils. The grout forms a bulb and thus displaces and compacts the surrounding soil. “Slab-Jacking” refers to injecting the thick mortar beneath a concrete slab so as to “level” it.

Jet Grouting – Jet grouting is a process in which a high-pressure water jet is used to erode the native soil and mix it or replace it with a stabilizer such cement or bentonite. The grout-soil mixture forms high strength or low permeability columns, panels or sheets, depending on the orientation and rotation of the jets, as they are withdrawn from the ground. Consequently, jet grouting is the precursor to deep soil mixing and is akin to forming weak auger cast piles. Columns of up to about 1 m diameter are typical, although much larger columns are possible using special equipment. Jet grouting can be used in most soil types, although it works best in soils that are easily eroded, such as cohesionless soils. Cohesive soils, especially highly plastic clays, can be difficult to erode and can breakup in chunks. A drawback of jet grouting is that it is very expensive and that special equipment is required. However, one advantage is that treatment can be restricted to the specific layer requiring improvement.

Fracture Grouting – Is the most recent grouting technology introduced into the USA in the 1990's. Essentially it is precision "slab-jacking" whereby settled structures are restored to their original elevation in a highly controlled fashion and the bearing capacity increased. It is used primarily in consolidating clayey soils not penetrable by grouts. Soil fracture grouting requires that the soil be fractured, not permeated. Cement or chemical grouts may be used.

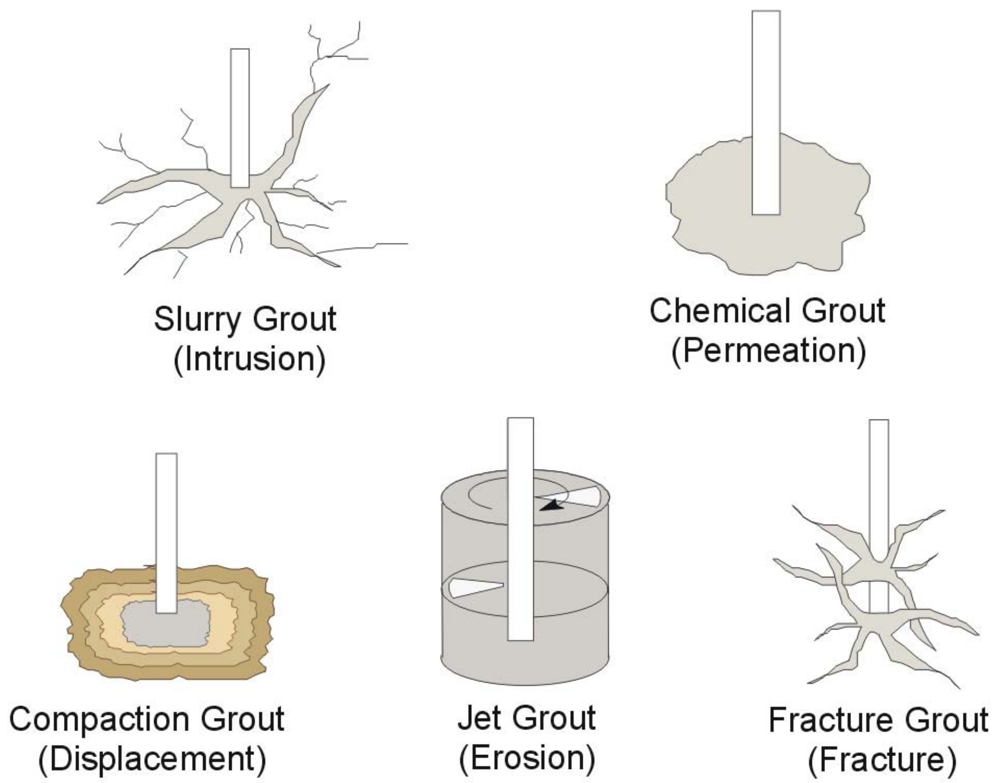


Figure 7.1 Basic Types of Grouting (from Hayward Baker)

Grout Materials

There are 2 fundamental grout types: (1) cement based suspensions, and, (2) chemical.

Cement-Based Grouts (FHWA,1998)– Slurry grouting uses grout composed of cement, water, and a small amount of bentonite clay (2 – 4%). Since portland cement is insoluble and settles rapidly from suspension, the small amount of clay prevents "bleeding" and keeps the cement in suspension. Fluidizing agents also may be added to reduce viscosity. Other additives may be added as fillers; e.g., sand, fly-ash, lime, etc. Water: cement ratios are generally 1_{H2O}: 2_{PC}

Type I Portland cement – most common

Type II Portland cement – resists sulfate attack and slower heat of hydration than type I

Type III Portland cement – high early strength

Type IV Portland cement – generates less heat of hydration and slower strength gain

Type V Portland cement – resists severe sulfate attack

Microfine cements – MC –100, MC – 300, and MC – 500 are finely ground cements for penetrating finely fractured rock or fine sands.

Chemical Grouts (FHWA, 1998, US ARMY COE, 1995) – Chemical grouts were developed to provide strength and control water flow in geologic units where the pore sizes in the rock or soils were too small to allow the introduction of conventional portland-cement suspensions. The first grouts used were two-stage grouts that depended on the reaction between solutions of metal salts and sodium silicate. The intent was to bond the particles of soil or rock and to fill in the pore spaces to reduce fluid flow. Grouting technology has expanded with the addition of organic polymer solutions and additives that can control the strength and setting characteristics of the injected liquid Typically, for water control acrylamides (not recommended), acrylates, silicates, lignosulfates, and polyurethane are used. Whereas, for strength enhancement, sodium silicates are used. The five basic chemical grouts are: (US Army COE,1995) sodium silicate, acrylate, lignin, urethane, and resin grouts

Sodium Silicate Systems – Sodium silicate grouts are the most popular grouts because of their safety and environmental compatibility. Sodium silicates have been developed into a variety of different grout systems. Almost all systems are based on reacting a silicate solution to form a colloid, which polymerizes further to form a gel that binds soil or sediment particles together and fills voids. Sodium silicate solutions are alkaline, and as this alkaline solution is neutralized, colloidal silica will aggregate to form a gel if the sodium silicate is present in concentrations above 1 or 2 percent (by volume). Typical alkaline reactants used are: (1) Acid reactant (phosphoric acid, sodium hydrogen sulfate, sodium phosphate, carbon dioxide solution). (2) Alkaline earth and aluminum salts (calcium chloride, magnesium sulfate, magnesium chloride, aluminum sulfate and (3) Organic compounds (glyoxal, acetic ester, ethylene carbonate formamide).

Acrylate Grouts (US Army COE, 1995) – Acrylates were introduced as less toxic alternatives to the toxic acrylamide compounds (AM-9) that are no longer available as grout. Acrylate grout is a gel formed by the polymerization of acrylates. The gelling reaction is catalyzed by the addition of triethanolamine and ammonium or sodium persulfate to a metal acrylate (usually magnesium acrylate). Methylene-bis-acrylamide is used as a cross linking agent. Potassium ferricyanide is used as an inhibitor if long times of setting are required. The acrylates have replaced acrylamide as the usual grout for forming water stops around sewer systems, and typically are not used in areas where it is subject to wetting and drying or freezing and thawing. Although not used primarily for strength acrylates typically form soft gels and sand samples grouted with acrylates can obtain strengths as high as 1.5 MPa. Acrylate grouts can be prepared with viscosities as low as 1 cP. The low viscosity and ability to develop long gel times (up to 120 min) make acrylate grouts useful in fine sediments.

Urethanes (US Army COE, 1995) – Urethane grouts are available in several different forms, but all depend on reactions involving the isocyanates cross-linking to form a rubbery polymer. One-part polyurethane grouts are prepolymers, which react with water to complete polymerization. The grouts will typically gel or foam depending on the amount of water available. Viscosities range from 50 to 100 cP. The two-component grouts employ a direct reaction between an isocyanate liquid and a polyol and produce a hard or flexible foam depending on the formulation. Viscosities range from 100 to 1,000 cP. Unfortunately, isocyanates typically have varying degrees of toxicity depending on the exact formulation. The solvents used to dilute and control the viscosity of the urethane prepolymers are also potential groundwater pollutants. Additionally, some grouts are highly flammable before and after setting. On the positive side, they can be injected directly into flowing water as a water stop and can be used for seal openings as small as 0.01 mm. Rigid foams have found applications in distributing loads in underground structures.

Lignins (US Army COE, 1995) – Lignin is a by-product of the sulfite process for making paper and when combined with a oxidizer such as sodium dichromate, it forms an insoluble gel in a short time. Lignins are generally not acceptable if chromium compounds are used due to the toxicity of chromium. A wide range of viscosities can be obtained which makes the lignins capable of being injected into voids formed by fine sands and possibly coarse silts. The materials used in lignin grouts are rapidly soluble in water, although mechanical agitation is recommended. The lignin gel in normal grout concentrations is irreversible, has a slightly rubbery consistency, and has a low permeability to water. Short-term observations (less than 2 years) show that for grouted materials protected against drying out or freezing, the grout will not deteriorate. Lignin grout is intended primarily for use in fine granular material for decreasing the flow of water within the material or for increasing its load-bearing capacity. These grouts have also been used effectively in sealing fine fissures in fractured rock or concrete. Their use in soils containing an appreciable amount of minus #200 sieve fines is generally unsatisfactory and not recommended because of unsatisfactory penetration. However, lignin grout of low viscosity injected at moderately high pressures may be effective in fine materials.

Resins (US Army COE,1995) – Resin grouts consist essentially of solutions of resin forming chemicals that form a hard resin upon adding a catalyst or hardener. The principal resins used as grouts are epoxy and polyester resins. Resins can be formulated to have a low viscosity; however, the viscosities are generally higher than those of other chemical grouts. Resins generally give off a large amount of heat during curing. They retain their initial viscosity throughout the greater part of their fluid life and pass through a gel stage just before complete hardening. The time from mixing to gel stage to hardened stage can be adjusted by varying the amount of the hardening reactant, by adding or deleting filler material, and by controlling the temperature, especially the initial temperature.. Epoxy resins, in general, exhibit compressive strengths greater than 70 MPa are attainable and may reach 270 MPa in a filled system (sand+epoxy). Tensile strengths generally range in excess of 28 MPa.

APPLICATIONS (FHWA, 1998, Mitchell, 1981)

Grouting is used for water control, and/or structural improvement. Because it is an insitu treatment it has an advantage over removal and replacement of poor soils. Additionally, it is less disruptive to surrounding areas. A variety of structural improvements are:

Desification – Whereas deep dynamic compaction, and vibro-compaction are pre-construction improvements, compaction grouting and soil fracturing as successful densification techniques for densifying loose granular soils beneath existing structures.

Settlement Mitigation and Restoration – Slab-jacking and soil fracturing have been used successfully to restore post-construction settlements. Pre-construction grouting fills voids to prevent future settlements.

Ground Strengthening – Grouting has been used to strengthen ground under existing structures to prevent settlements due to adjacent excavations, dewatering, etc. Strengthening can be used to provide lateral support for excavations or tunneling, and foundation underpinning.

Liquefaction Mitigation – Densification and strengthening of loose granular deposits to mitigate liquefaction potential.

SLOPE STABILIZATION

Sinkhole Remediation – Grouting has been widely used in Florida for sinkhole remediation.

Figure 7.2 Illustrates several grouting applications.

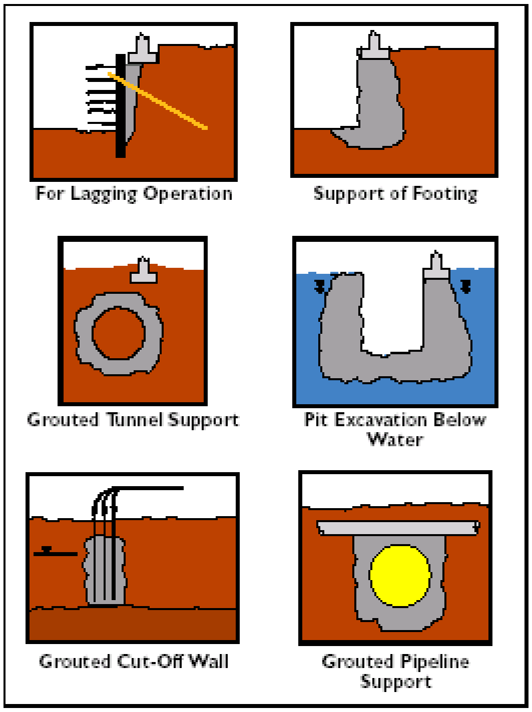


Figure 7.2 Illustration of Grouting Applications (from Hayward Baker)

GROUTING FEASIBILITY

The feasibility of grouting is typically dictated by the insitu soil type to be treated and cost. Of all the previously mentioned grout methods illustrated in Figure 7.1, only jet grouting is applicable to all soils. Figure 7.3 (Hayward Baker) shows the grout method vs. soil type compatibility.

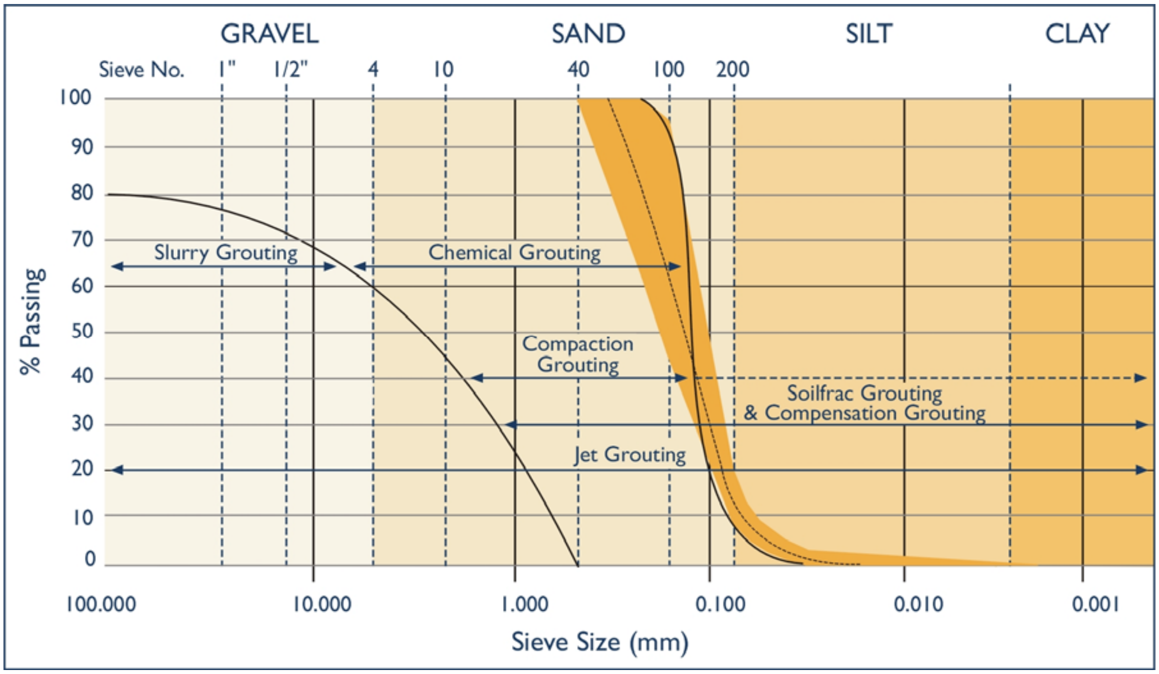


Figure 7.3 Range of Soil Grouting Techniques (from Hayward Baker)

The selection of grout material (cement particulate vs. chemical) obviously depends upon the void size penetrable by the grout. Void size is easily related to grain size distribution. As illustrated in Figure 7.4, the particulate cement grouts are limited to gravels and coarse sands. Alternatively, the chemical grouts can penetrate the smaller voids of the fine sands.

Slurry grouting is to seal rock cracks and fissures and coarse grained soils either for water control or strengthening. Mitchell (1981) presented "groutablility ratios" for use in estimating use of particulate cement grouts for coarse grained soils. For example, usually 95% of type I Portland cement will pass the No. 200 sieve thus exhibiting a D_{95} of 0.074 mm. Thus, the minimum groutable soil will have a $D_{10} > 11\ (0.074) = 0.8$ mm. Obviously, microfine cements with smaller particle sizes can penetrate finer soils. Typically microfine cement will exhibit a D_{95} of .007 mm, and thus the minimum groutable soil will have a $D_{10} > 11\ (.007) = .08$ mm or 10 times smaller than type I Portland cement. The grain size distribution for particulate cements are presented in Figures 7.5 and 7.6.

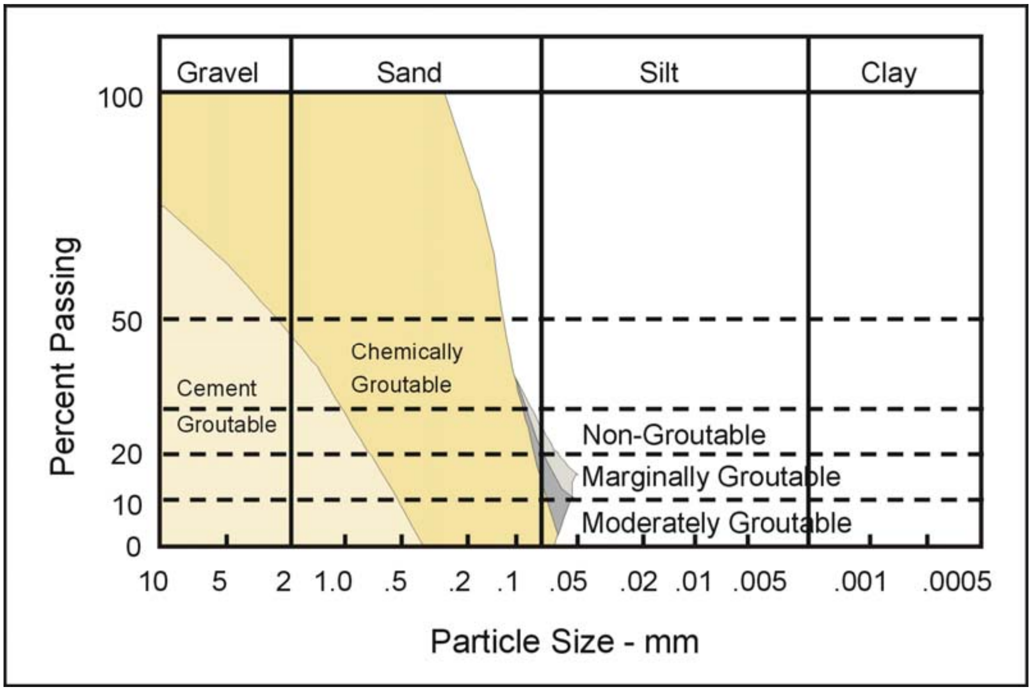


Figure 7.4 Soil – Grout Material Compatibility

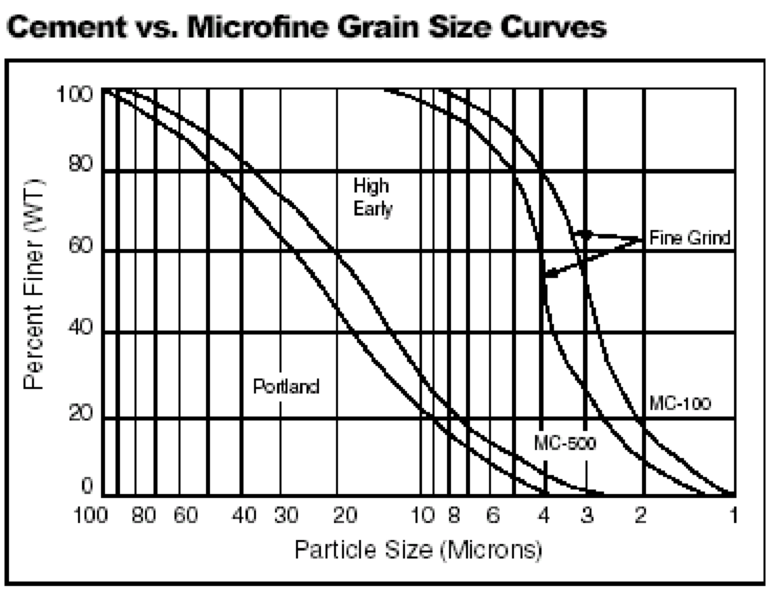


Figure 7.5 Grain-Size Distributions for Particulate Cements

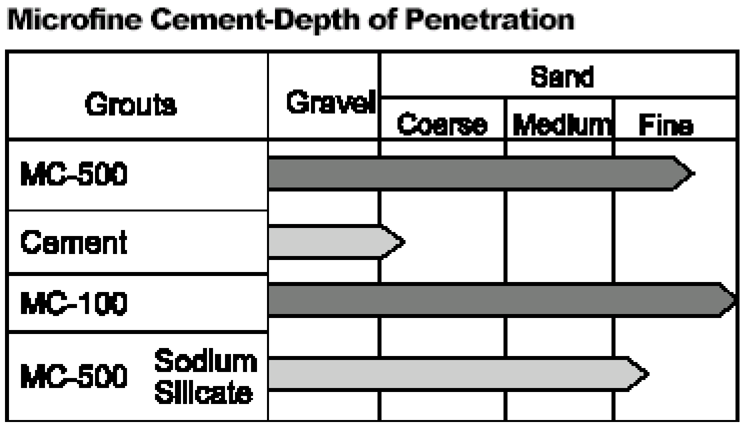


Figure 7.6 Illustration of Penetration by Particulate Cements

Groutability Ratios

$$\text{For Soils}: \quad N = \frac{(D_{16})\,\text{Soil}}{(D_{85})\,\text{Grout}}$$

$N > 24$: Grouting consistently possible
$N < 11$: Grouting not possible

$$N_c = \frac{(D_{10})\,\text{Soil}}{(D_{95})\,\text{Grout}}$$

$N_c > 11$: Grouting consistently possible
$N_c < 6$: Grouting not possible

$$\text{For Rock}: \quad N_R = \frac{\text{Width of fissure}}{(D_{95})\,\text{Grout}}$$

$N_R > 5$: Grouting consistently possible
$N_R < 2$: Grouting not possible

Additional guidelines relating to particular grout types and particle size are:

Types I and II Portland cement are suitable for soils coarser than 0.60 mm.

Type III Portland cement is suitable for soils coarser than 0.42 mm.

Chemical grouts offer an advantage over cement particulate grouts in that they can penetrate finer voids as illustrated in Figure 7.7. Usually soils containing less than 10 percent fines can be permeation grouted via chemical grouts, and soils with more than 20 percent fines will be impossible to permeate.

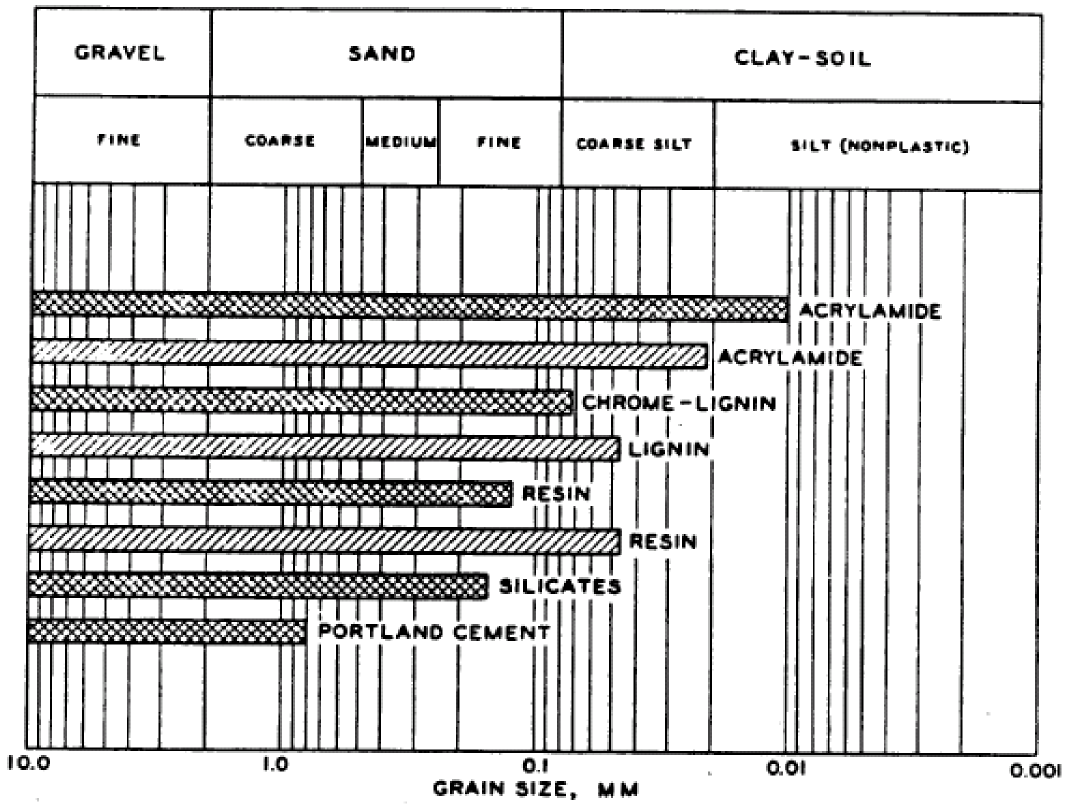


Figure 7.7 Injection Limits for Various Grout Materials (from EM1110-1-3506)

ENGINEERING CONSIDERATIONS

Assuming permeation grouting, the grout moves outward through the soil under pressure displacing air and water in the voids as dictated by the soil's permeability (Xanthakos,1994). The engineering questions then are:

What is the injection pressure?

What distance will the grout flow?

What is the time required?

Injection Pressures – It has been widely assumed that a "rule of thumb" is that the grouting pressure not exceed the total overburden pressure overlying the zone being grouted. (Weaver,1991). This translates for "typical" soil unit weights to 1psi per ft. of depth. However, the Corps of Engineers notes that geological factors, rock strength, and extent of rock fracturing, affect grouting pressures. Conversely, European practice uses a "rule of thumb" of 1 kg/cm^2 per meter of depth, which is about four times that used in the United States. Figure 7.8 illustrates US and Swedish grouting practices. (Weaver,1991)

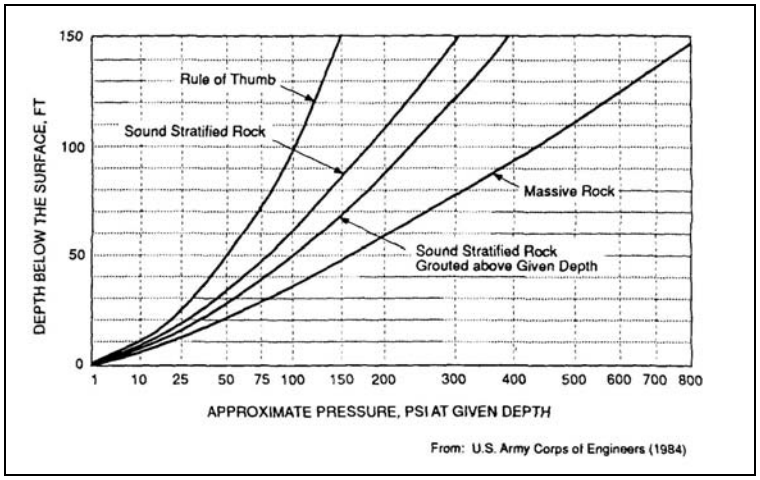


Figure 7.8a US Grouting Practice Pressures

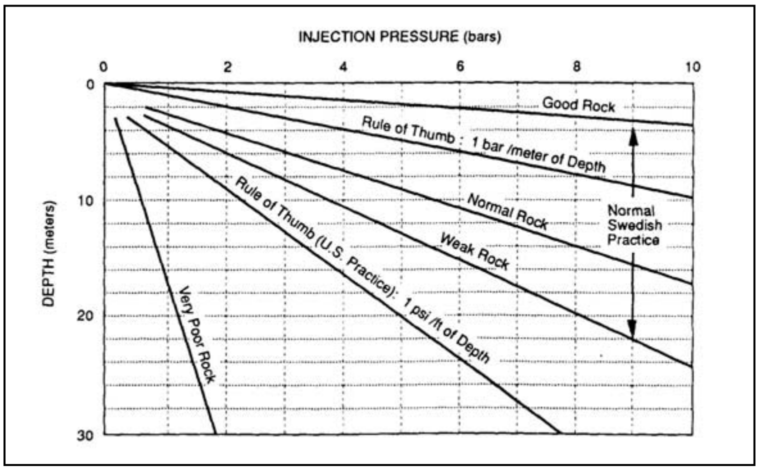


Figure 7.8b Swedish Grouting Practice Pressure

From US Army COE (1995), the equations for sound stratified and massive rock, respectively in Figure 7.8a are:

$$P(\text{psi}) = h + 1.33h\left(\frac{h}{900} + \frac{\sqrt{h}}{20}\right),\ h = \text{ft.}$$

$$P(\text{psi}) = h + 1.33h\left(\frac{h}{100} + \frac{3\sqrt{h}}{20}\right),\ h = \text{ft.}$$

Grouting Distances (Xanthakos,1994) – In uniform isotropic soils, spherical flow and Darcy's Law are assumed, and the radius of penetration, R, is related to the rate, Q (vol/time), and hydraulic head, H, as:

$$H = \frac{Q}{4\pi k}\left[\mu\left(\frac{1}{r}+\frac{1}{R}\right) + \frac{1}{R}\right]$$

where:

k = soil permeability

μ = grout viscosity

r = radius of grout tube

The time for the grout to penetrate to radius R is:

$$t = \frac{\mu n}{3kHr}(R^3 - r^3)$$

where: n = porosity

Alternatively Hausmann (1990) gives:

$$t = \frac{4\pi n}{3Q}\left(R^3 - r^3\right)$$

Thus, from these theoretical considerations, the penetration radius of the grout depends upon:

Permeability, k, of soil and rock

Injection pressure

Injection rate

Grout characteristics (setting time, viscosity)

Using the above equations, grouting patterns and spacings based upon R can be estimated, as shown in Table 7.1. (AFTES,1991)

Grouting Procedures (Mosely, 1993, Weaver, 1991)

Once the grout has been selected and hole pattern established, grout holes are drilled/ jetted to their appropriate depth, and grout is injected by, either:

1. Through an injection pipe in an open hole that is sealed (caulked) at the surface
2. Through an injection pipe held in place by a packer
3. From a grout pipe withdrawn as injection proceeds (bottom up grouting)
4. Through the grout pipe left in place as tube a' manchette

Since grouting rates are not large, boreholes are typically 1- to 5-inches, and spacings vary considerably from 2 to 10 ft. (See Table 7.1).

Table 7-1 Typical Drill Hole Spacings

Medium to be Injected	Description	Distance Between Holes (m)
Soil, depth < 25 m	Fine sand	0.8-1.3
	Sand, sand and gravel	1.0-2.0
	Gravel	2-4
	Gravel	
	Sand and gravel	Watertight ground 3-5
	(kH > kV)	
Rock depth < 25 m	Fine cracks	1-3
	Open cracks	2-4
Structures	Backing behind the vault	2-3
Cavities	Filling of large voids	3-15

Source: from AFTES (1991).

Drilling may be either rotary or percussion, although percussion drilling is more commonly used. Flushing of rock cuttings is done by air or water, with water being more common. Descending (top of hole downward) or ascending (bottom of hole upward) grouting is dependent upon geological and economical factors. Ascending is less expensive and more popular provided smooth holes and good seals can be obtained with packers. In ascending grouting the hole is drilled to full depth and a packer placed at the top of the lowermost grouting stage. When this stage is grouted to refusal, the packer is raised to the top of the next stage. In descending grouting, the hole is drilled to a shallow depth, and grouting of that stage proceeds. The grout is usually washed out of the hole before it takes a final set, so that redrilling to the subsequent deeper stage is minimized. Obviously descending grouting is more cumbersome and expensive than ascending; hence descending may only be used to seal the upper stages and then ascending grouting is used of the remainder of the hole. Figure 7.9 (Xanthakos,1994, from Ewart,1985) illustrates these methods.

Refusal Criteria (Weaver,1991) – Refusal criteria are indicative of the reduction in grout takes needed to provide significant permeability reductions. Several criteria have evolved.

1. U.S. Army Corps of Engineers criteria required that grouting continue until the hole “takes” grout at the rate of 1 ft^3 or less in 10 minutes measured over a 5-minute period. Alternatively, grouting continues until no grout is taken at 75% of the maximum grouting pressure.
2. USBR criteria recommends that grouting continue until a grout take of less than;
    a. 1 ft^3 or less in 20 minutes for pressures < 50 psi
    b. 1 ft^3 or less in 15 minutes for pressures 50 – 100 psi
    c. 1 ft^3 or less in 10 minutes for pressures 100 – 200 psi
    d. 1 ft^3 or less in 5 minutes for pressures > 200 psi

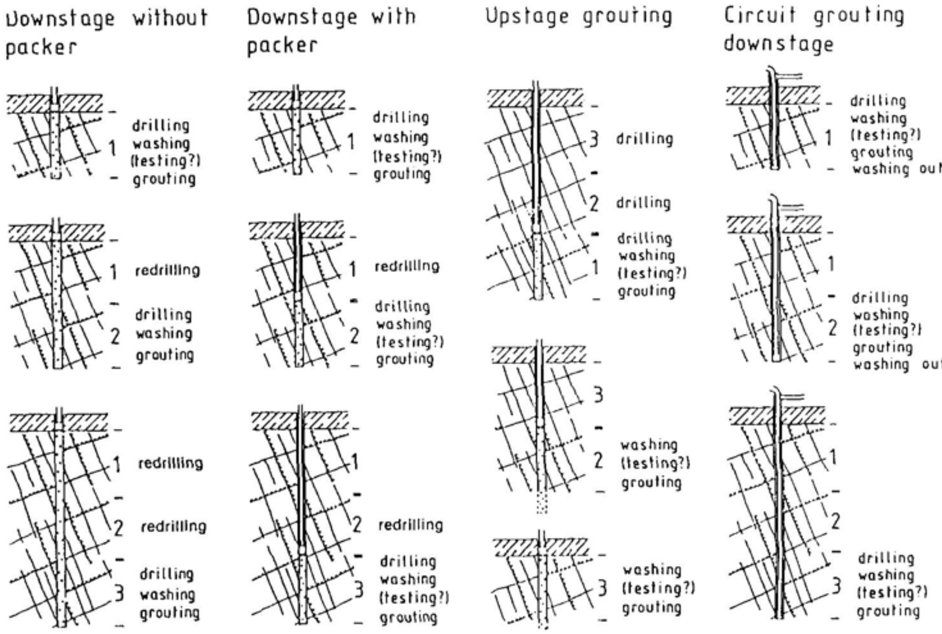


Figure 7.9 Stage Grouting Methods for Rock Grouting (from Xanthakos, 1994, Ewart, 1985)

Diagnostics of Observed Pressure Takes (Weaver, 1991) – By periodically monitoring the injection rates at 10- to 15-minute intervals, trends in grouting can be discerned. Table 7.2 (Weaver, 1991) illustrates diagnostic trends and if appropriate action is warranted.

Table 7-2 Interpretation of Pressure-Take Trends

Pressure-Take Trend	Interpretation and Remarks	Corrective Measures
Constant pressure and gradually decreasing grout take.	This trend indicates gradual plugging of fractures. Common with low to moderate takes in rock with many small fissures.	No correction generally required. May control grout travel by cautiously adjusting the pressure or the mix, if desired.
Constant pressure and gradually decreasing grout, followed by sudden decrease in take.	Premature plugging.	Inject water, resume grouting with thinner mix. Subsequent rate of take should not be higher than that noted prior to plugging. Caution should be exercised to avoid uplift while injecting water or thin grout.
Constant pressure and constant moderate grout take for long period of time at specified pressure.	Grout flow to surface or to open holes, or extensive grout spread. Common trend in primary holes.	If no surface leaks, thicken mix gradually. If no decrease in rate after reasonable number of bags injected. Inject water and allow grout to take initial set. Regrout later.
Constant pressure and slowly decreasing or constant large take followed by sudden increase in take.	Local rock displacement, or widening of passages in the rock without displacement. Knowledge of adjacent geologic conditions essential in assessing this trend.	Thicken the mix and reduce the pressure until the grout take changes to a decreasing trend. Check survey points or displacement gages.

Pressure-Take Trend	Interpretation and Remarks	Corrective Measures
Constant pressure and quickly decreasing take followed by gradual decrease.	Local, probably confined, voids are filled first, followed by a gradual filling of line fractures.	A fluid mix should be used. Increase superplasticizer dosage if using stable grout.
Low pressure and constant large grout take at maximum pump capacity, using thick mix.	Free flow of grout into heavily broken rock. Prolonged grouting would result in wide grout spread, and wastage of materials.	Add filler to grout. Suspend grouting after reasonable number of bags injected; inject water and regrout later from same or adjacent hole.
Low or slow rising pressure and decreasing take at maximum pump capacity.	Limited grout travel in moderately broken rock. Common trend in primary holes. Specified pressure will be reached when the pump capacity exceeds the rate of grout take.	Stop thickening mix, and continue grouting to refusal.
Rapid increase in pressure and decrease in take.	Premature plugging due to thickening mix too rapidly.	Inject water and fluidity mix.
Decreasing grout take gradually changes to increasing take. Use of thicker mix has no effect.	Gradual rock displacement over large area or scouring of a channel in a filled large fracture.	Decrease pressure, check survey points or displacement gages.
Erratically decreasing trend, with fluctuations in both pressure and take.	Gradual plugging of fractures in broken or slabby rock.	None required.
Erratically constant trend. Heavy pressure and take fluctuations with no permanent increase or decrease.	Rock slabs, blocks, or places are locally displaced by grout opening new passages, frequently changing the resistance to grout flow. Usually associated with heavily broken rock and large grout takes.	Thicken mix fairly rapidly until decreasing trend is noted until maximum practicable thickness of mix is reached. Avoid excessive grout travel by slopping grouting after reasonable number of bags injected. Regrout later.

Borehole Pump Tests (Lugeon Test) – A borehole pump test may be performed to evaluate the effectiveness of the grouting program. The test consists of isolating a section of a borehole using packers and injecting water under varying pressures into the grouted mass. The flow of water is measured at 10 minute intervals under a constant pressure. The test is performed in five stages. A initial low pressure is applied followed by a medium pressure, and lastly a peak pressure of 10 tsf.; and then rebounded to the medium and low pressures. For a particular test, the water take in Lugeons is:

$$Water\ take\ (Lugeons) = water\ take\ in\ liters/meter/\min. \times \frac{10\ bars}{test\ pressure\ (bars)}$$

Alternatively, a borehole pump test may be reported as "fluid conductivity value" (rkf), where;

$$\text{rkf} = \frac{Q}{2\pi Lh} \ln \frac{L}{r}$$

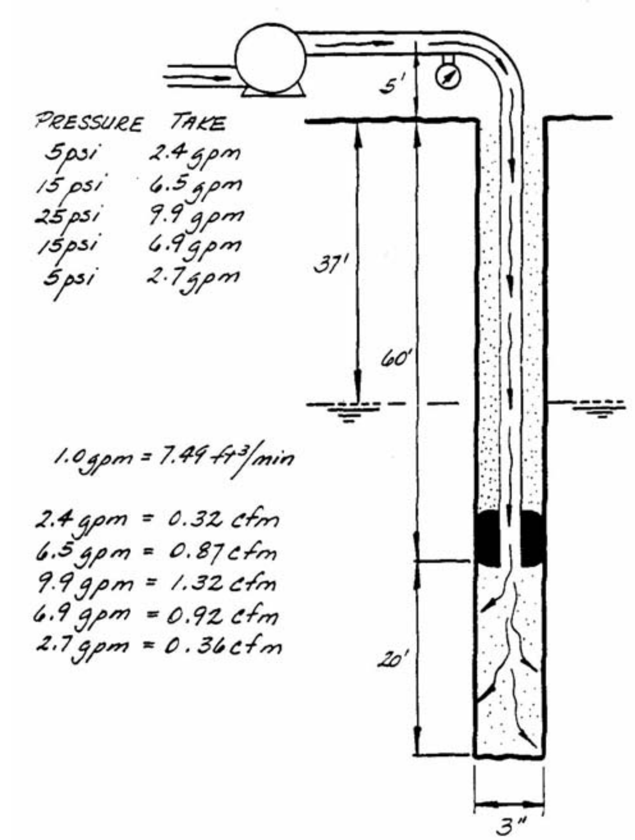
Q = rate of fluid flow (Vol/time)

L = length of borehole tested

H = head including elevation and pressure

R = borehole radius

<u>Example (USACE)</u>: A stage test is being performed on a 20-ft. (6.1 m) zone of rock at 60 to 80 –ft. below ground surface. The ground water table is located at 37-ft. below ground surface. The pump gage is located 5-ft. above ground surface and the borehole diameter is 3-inches. Find the Lugeon and rkf values when the pressure is 25 psi, and water take = 9.9 gpm.



Vol /minute =

$$9.9\ \text{gpm}\ \times \frac{3.78\ \text{L}}{1\text{gal}} = 37.4\ \text{L/min}$$

$$\Delta H = 5\text{ft} + 37\ \text{ft} = 42\ \text{ft x}\ \frac{1\text{psi}}{2.31\ \text{ft}} = 18\ \text{psi}$$

H = 25 psi (gage) + 18 psi (ΔH) = 43 psi

$$\text{Water take (Lugeons)} = \frac{981 V_W}{p_e L t} = \frac{1000 V_w}{p_e L t}$$

$$\text{Water take (Lugeons)} = \text{water take in liters/meter/min.} \times \frac{10\ \text{bars}}{\text{test pressure (bars)}}$$

$$\frac{37.4\ \text{L}}{6.1\ \text{m}} \times \frac{10\ \text{bars}}{43\ \text{psi}} \times \frac{15\ \text{psi}}{\text{bar}} = 21\ \text{lugeons}$$

$$\text{rkf} = \frac{Q}{2\pi L h} \ln \frac{L}{r} \qquad r = \frac{3''}{2} \times \frac{1}{12} = 0.125\ \text{ft} \quad \text{and} \quad Q = 9.9\ \text{gpm} \times \frac{1\ \text{gpm}}{7.49\ \text{cfs}} = 1.32\ \text{cfm}$$

h = 43 psi (see above lugeons) x 2.31 ft/psi = 99.3 ft

$$\text{rkf} = \frac{1.32\ \text{cfm}}{2\pi(20\ \text{ft})(99.3\ \text{ft})} \ln \frac{20\ \text{ft}}{0.125\ \text{ft}} = 5.4 \times 10^{-4}\ \text{ft/min}$$

One Lugeon is approximately equivalent to a permeability of 10^{-7} m. Hausmann (1990) credits Lugeon with recommending groutability criteria. If a dam foundation exceeds 1 lugeon (heads > 30 m) or 3 (heads < 30m), then a grout curtain is needed.

Compaction Grouting of Sinkhole (from Hayward Baker, Fuleihan,1996)

On June 27, 1994, a very large sinkhole was discovered during a routine inspection of an inactive phosphogypsum disposal stack, at the IMC-Agrico Co. concentrated phosphate plant in Polk County, Fla. The resulting remediation project cost $6.8 million, including $1.2 million for preliminary exploration and $5.6 million for sinkhole repair.

The sinkhole was 160 ft in diameter across the top and tapered to a 110 ft wide shaft that extended beyond the sedimented gypsum base located 200 ft beneath the surface—making it one of the largest sinkholes of its kind. Because it was located in a disposal area used to store phosphogypsum, a by-product of the concentrated phosphate manufacturing process that contains acidic water, it presented major structural design as well as environmental challenges. The sinkhole caused ponded water and seepage water from the stack—as much as 2—6 million cu ft—to flow into the underlying Floridian Aquifer.

The subsurface exploration program overcame serious difficulties related to providing safe access to the erosion cavity by using angle drilling from the edge of the scarp. A cross-hole seismic survey was conducted to assist in defining the location and extent of the cavity in the confining unit beneath the stack. (See Figure 7.9)

Remediation involved injecting nearly 4,000 cu yd of concrete 400 ft beneath the surface through 50 grout-injection casings. By using multilevel phased angle drilling and grouting sequences the structural integrity of the confining unit was restored, and any further leakage of pore water from the phosphogypsum. sum stack was eliminated.

More than 100 grout mixes were tested to obtain a special mix that was pumpable, would not segregate or bleed, was compatible with acidic pond water, and would exhibit the desired strength and hydraulic conductivity over a wide range of slumps. The primary pea gravel concrete grout contained; aggregates, fly-ash, Type II cement, bentonite, water, and a plasticizer to maintain strength at high slumps. The second liquid grout contained; fly-ash, Type II cement, bentonite, and a plasticizer. Both mixes were rich in cement. Because of the corrosive nature of the pore water, steel casings had a very limited life span. Round-the-clock operations were planned and coordinated to expedite the repair work and to preclude grout from flowing from an injection casing toward another hole that was still being advanced or that had not up to that point been grouted. This task was particularly challenging because of the very limited work area and the close proximity of a large number of inclined casings.

Maximum flexibility was required to allow for changing drilling/grouting procedures and grout mixes to accommodate changing conditions. An on-site concrete batch plant and ready-mix trucks were mobilized to provide added flexibility.

The erosion cavity in the confining unit was infilled with cemented gypsum blocks that had collapsed from the stack. Concrete was injected at high pressure not only to fill the voids but also to cause the grout to flow via hydraulic fracturing. Because of the intimate bonding between the concrete and gypsum, the resulting “gypcrete” exhibited excellent strength and hydraulic properties.

A multistep drilling methodology was implemented to advance each of the 50 grout casings to inclined lengths of up to 450 ft. Three casing sizes (9, 6.75, and 4.5 in. nominal diameter) and two types of rigs were used. Precise angle drilling was required to ensure that the grout casings were terminated within the cavity at the proper elevations. A Fotobore directional survey performed inside the casings determined precise bearing and inclination. Any deviation from the target inclination and bearing of a casing had to be accounted for in planning future grout holes to achieve complete grout coverage. Approximately 3800 yds^3 of pea gravel concrete, were injected 400 ft beneath the surface for restoration.

In an upstage grouting sequence, each casing was slowly extracted while grouting operations were in progress. A vibratory hammer assisted in the extraction process so as to minimize the potential for the inclined casing getting stuck in grout Phased grouting operations progressed from deeper to shallower target levels in the confining unit. The first objective was to seal the throat of the cavity in order to minimize grout losses.

Extensive monitoring of the water table and of piezometric levels determined the progress and effectiveness of the remedial work aimed at plugging the erosion cavity.

Grouting operations began in December 1994. By late February 1995, water began filling the cavity, indicating that the hole had been plugged, and grouting under high pressure was continued until April 1995. There has been no impact on ground-water resources beyond the plant site. The remediation filled the cavity and plugged the hydraulic connection between the gypsum stack and the Floridian Aquifer.

Contaminant levels in the plant-production wells have since exhibited a systematic downward trend. Ground water and potentiometric surfaces have returned to presinkhole levels. After the cavity was remediated, the hole in the stack was refilled with sedimented gypsum, restoring the stack’s permeability and original appearance.

Figure 7.9 Photo of Sinkhole

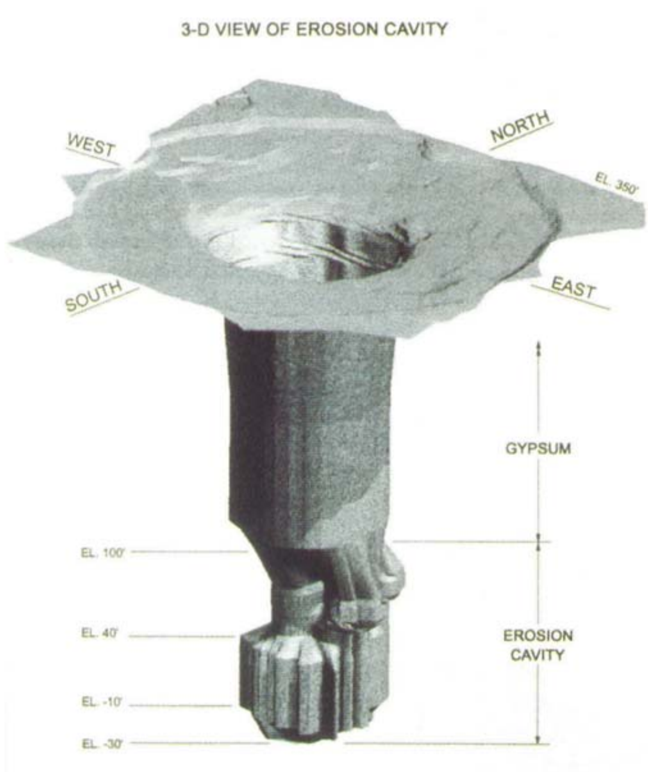


Figure 7.10 View of Erosion Cavity

REFERENCES

AFTES, 1991. "Recommendations on Grouting for Underground Works," Tunneling and Underground Space Technology, Vol. 6, No. 4, reported in Xanthox.

Ewart F.,1985. *Rock Grouting*, Springer-Verlag, NY

FHWA (1998). ***Ground Improvement Technical Summaries, Final Draft,*** FHWA Demonstration Project No. 116, Contract No. DTFH61 -93-R-00 145, Elias, V., Welsh, J., Warren J. and Lukas R.

Fuleiham, N.F. and Rubin, G.J. (1996). "Sinkhole Remediation at a Phosphogypsum Stack," AIChE 1996 Spring Meeting, New Orleans, LA.

Hausmann, M.R. (1990). ***Engineering Principles of Ground Modification,*** McGraw-Hill, USA 632 p.

Hayward Baker (1996). ***Ground Modification Seminar Notes.*** Hayward Baker, Odenton MD. www.haywardbaker.com

Mitchell, J.K. (1981). "Soil Improvement: State-of-the-Art." State-of-the-Art Report, Session 12, 10th ***International Conference on Soil Mechanics and Foundation Engineering,*** Stockholm Sweden.

Moseley, M.P. (1993). *Ground Improvement*, Blackie Academis & Professional Publ CRC Press, Boca Raton, FL.

USAEC, Earth and Rockfill Dams Course Notes, Waterways Experiment Station, Vicksburg, MS.

U.S. Army Corps of Engineers (1995) EM 1110-1-3500 Chemical Grouting, Washington, DC 20314.

U.S. Army Corps of Engineers (1984) Engineer Manual 1110-2-3506 Grouting Technology, Washington, DC 20314.

Weaver, K. (1991) *Dam Foundation Grouting* ASCE Publ. Ruston, Va.

Xanthakos, P.P., Abramson, L.W. and Bruce, D.A. (1994). Ground Control and Improvement, John Wiley & Sons, New York NY, 910 p.

Chapter 8

JET GROUTING

INTRODUCTION

Experience has shown that cement grouting is limited to coarse grained cohesionless soils, and permeation chemical grouting, likewise, is limited to fine sands. Consequently, jet grouting offers an alternative to these grouting methods and is applicable to a wide range of soils (Figure 8.1). Jet grouting utilizes high pressure erosive jets to disaggregate soil particles, remove some of them, and subsequently mix the remaining particles with grout to form "Soilcrete" (Moseley, 1993). The concept was developed in the early 1970's and has found a niche for underpinning and excavation.

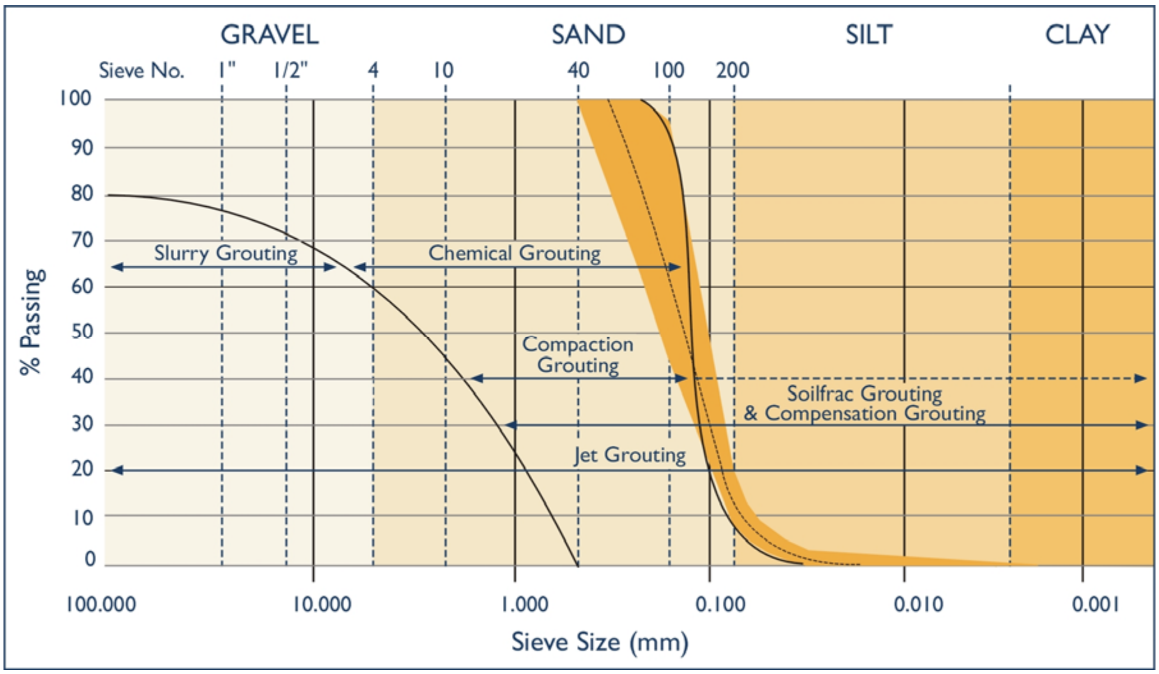


Figure 8.1 Range of Soils Suitable for Jet Grouting (from Hayward Baker)

Essentially, jet grouting consists of drilling a 6-inch borehole to a desired depth, jetting to erode the soil and place slurry, and gradually move the process upward as illustrated in Figure 8.2. Since jet-grouting is a bottom up method, the consistent jetting lifting operation creates a column expelling the eroded soil and any slurry used to maintain the borehole at the surface.

The grout used consists of Portland cement, bentonite, water, and fluidizers. Water/ cement ratios as low as 0.6 have been used where strength is paramount, and up to 2.0 for low permeability applications (Moseley, 1993).

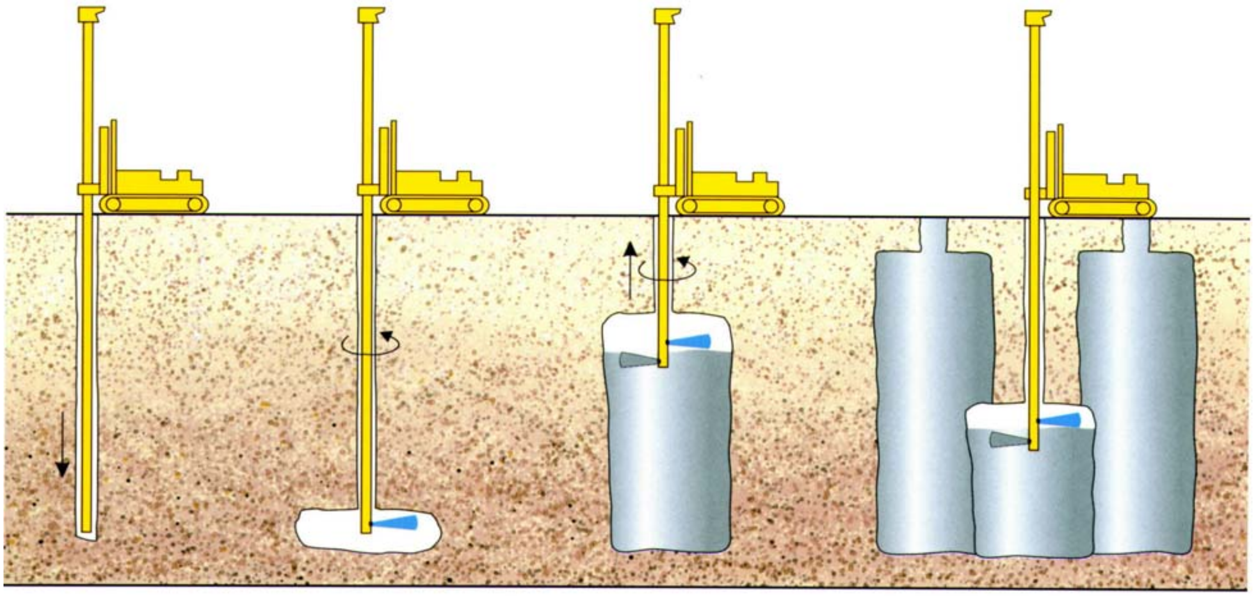


Figure 8.2 Schematic of Jet Grouting (from Hayward Baker)

Essentially, three jet systems have evolved, all relating to the type of erosive jet: (a) single rod system, (b) double rod system, and (c) triple rod system. (See Figure 8.3) (Hayward Baker, Moseley, 1993, Brill, 2003)

a. Single rod system – This is the simplest and earliest (1970's) system and incorporates a hollow stem grout pipe with one or several 2.0 – 4.0 mm jets using pressures of 600 bars and velocities of 200 m/s (650 fps) to create soil-grout columns of 0.1 – 1.2 m (2-4 ft). Single rod jet grouting is less effective in cohesive soils than cohesionless soils.

b. Double rod system – The double rod system is more advanced than the single rod and the erosive effect of the jet is enhanced by a compressed air shroud, typically between 2 – 15 bars. Two hollow stem pipes, the outer carrying compressed air, while the inner one carries the grout, are used to erode and form the soil-grout column. Columns more than 1.0 m (3 ft.) are possible in medium to dense soils, while columns more than 1.5m (5 ft.) diameter can be obtained in loose soils. The double rod system is more effective in cohesive soils than the single rod system.

c. Triple rod system – The triple rod system incorporates an air shrouded water jet for soil erosion similar to the double rod system, but adds a separate nozzle for injecting the grout. Consequently, the erosion process is separated from the grouting and yields a higher quality soil-grout "soilcrete." Water pressures up to 500 bars in conjunction with air pressures of 2 – 15 bars are used. Grout pressures do not need to be high and typically are between 5 – 30 bars. (Moseley, 1993). Columns from 0.9m (3 ft.) to 1.4m (4.5 ft.) can be achieved. Triple rod jet grouting is the most effective system for cohesive soils. (Hayward Baker)

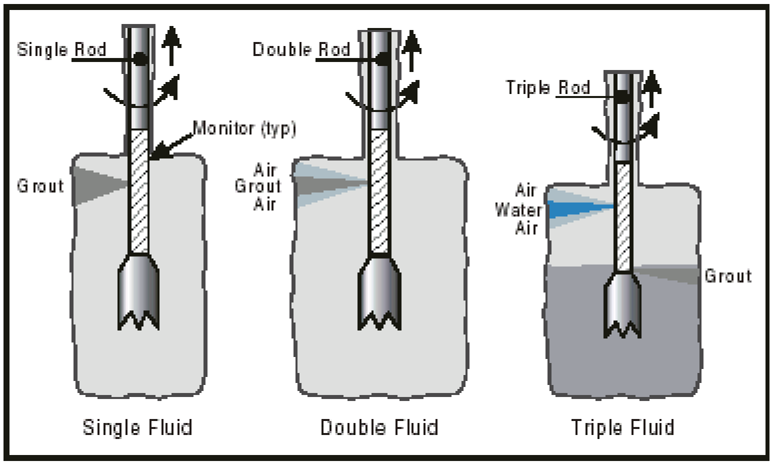


Figure 8.3 Jet Grouting Systems (from Hayward Baker)

d. Super Jet Grouting – The newest system of jet grouting entered the US from Japan in 2000. (Brill,2003, Hayward Baker). It is a two fluid system using opposing grout injection nozzles shrouded by compressed air (similar to the double rod system). High-quality large diameter (3.3 – 4.8 m , 11 – 16 ft) soil-grout columns can be created. Super Jet is applicable to all soils and is best applied for bottom seals and "surgical" treatment applications. (Hayward Baker). Figure 8.4 illustrates the method.

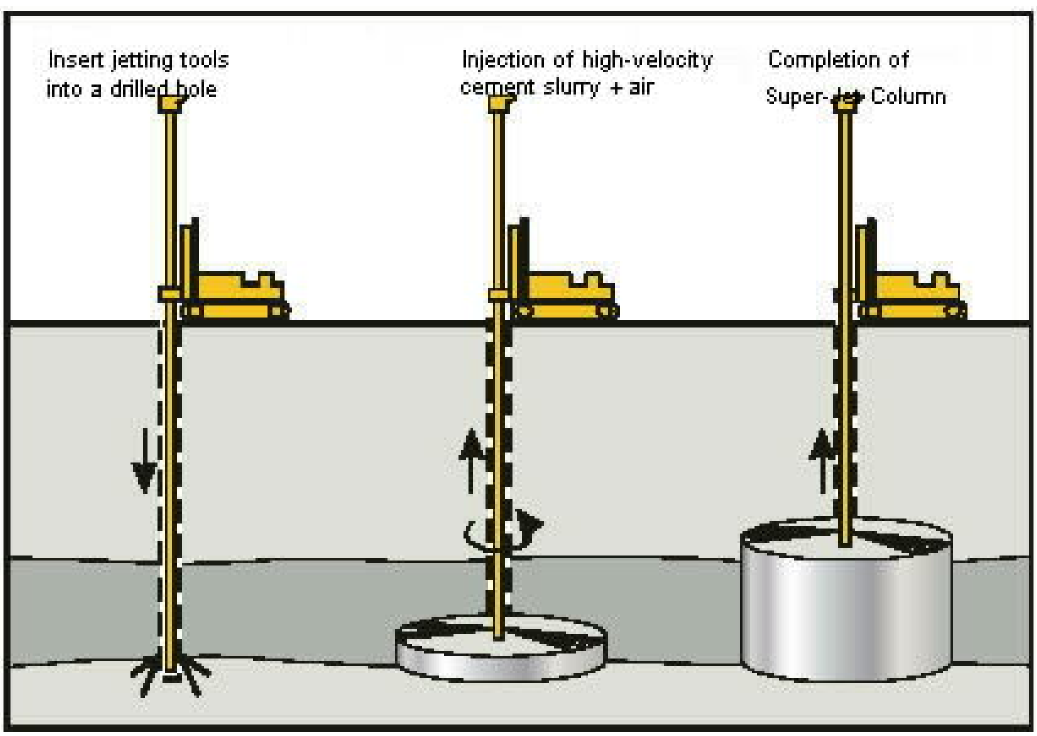


Figure 8.4 Super Jet Grouting System (from Hayward Baker)

APPLICATIONS/ADVANTAGES OF JET GROUTING (Brill, 2003)

Jet grouting advantages include:

a. All work can be accomplished in situ from the ground surface

b. Offers an alternative to conventional grouting, chemical grouting, deep slurry trenching, proprietary underpinning systems, or the use of compressed air or freezing in tunneling, etc. (See Figure 8.5)

c. Wide variety of configurations; i.e., columns, slots, etc. can be created. (See Figure 8.6)

d. Positive contact with foundations for underpinning can be achieved.

e. Can be performed under conditions of limited headroom or confined spaces.

f. Historical buildings can be underpinned with minimal settlements.

g. Large areas and volumes of problem soils can be treated.

h. Applicable to nearly all soil types

i. Vibration minimal

SOIL COMPATIBILITY (Moseley, 1993)

Although jet grouting is amenable for a wide range of soils, the influence of soil grading, structure, and state are important. Generally, cohesionless sands are best suited to jet grouting in that they are easily eroded and transported in the slurry to the surface. Consequently in these soils, large diameter columns or panels can be formed. Relative density is usually more important than gradation. Miki (1984) reports no effect of gradation when the uniformity coefficient is greater than 10.

Gravelly soils are amenable to treatment, but highly permeable poorly graded gravels may lose grout, thereby affecting intended geometries. Also large particles may shadow surrounding soil from the erosive jet diminishing jet efficiency and column diameter. Likewise mixing with grout or transportation to the surface in the waste slurry may be limited.

Even small amounts of cohesive affect the erosive jet action. Consequently, although silts and silty sands are well suited for treatment, the grout diameters cut may be smaller than clean sands. Cohesion effects are much more pronounced in clays and cohesive silts, and grout columns are dependent upon soil shear strength. When shear strengths exceed 50 kPa, the volumes of ground influenced by standard jet grouting procedures become too small to be practical.

PROPERTIES OF SOIL – GROUT COLUMNS

Table 8.1 summarizes typical properties for jet grouted columns (Moseley, 1993, FHWA, 1998). Figure 8.7 provides guidance of typical jet grout strengths.

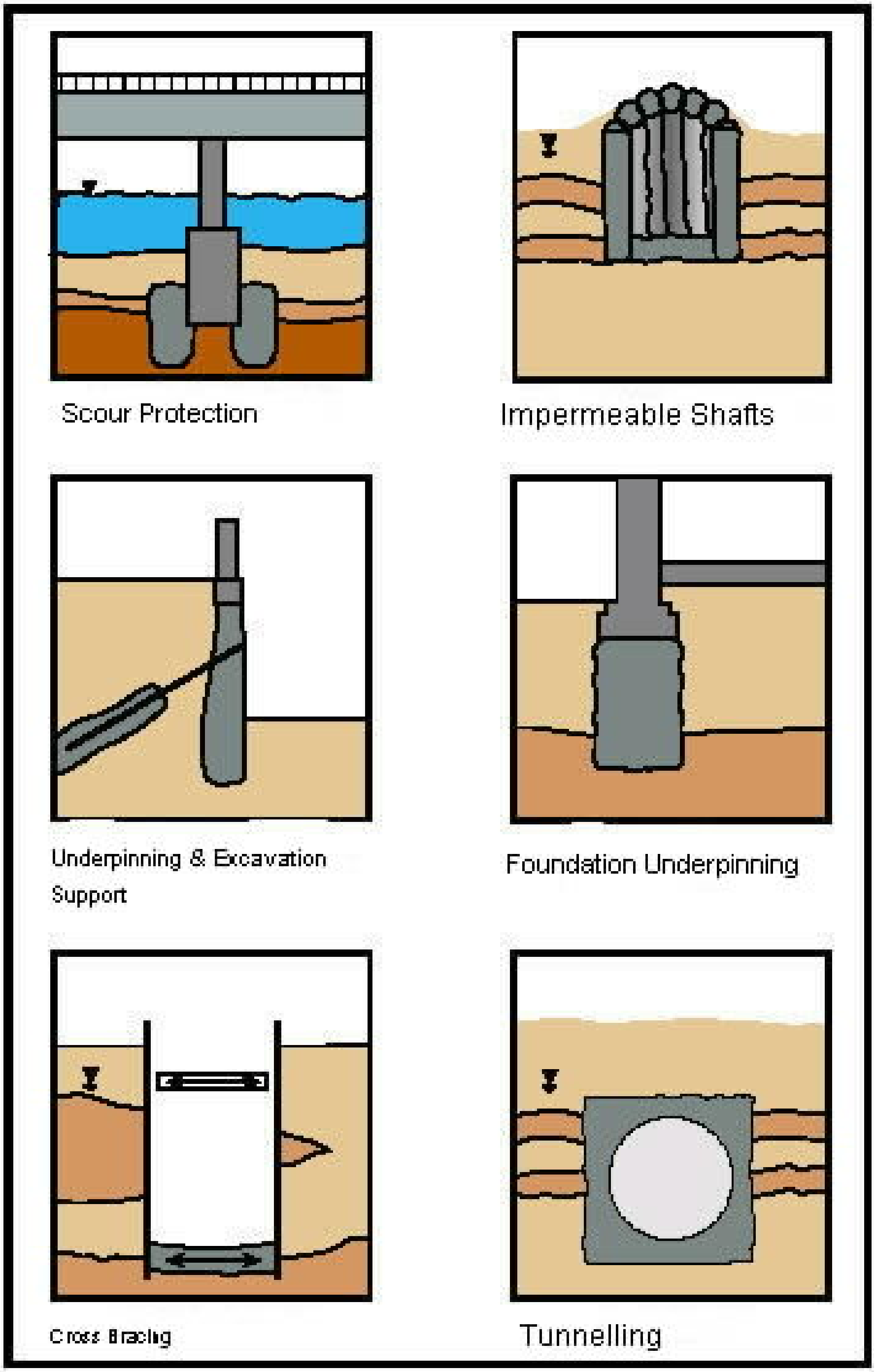


Figure 8.5 Applications of Jet Grouting (from Hayward Baker)

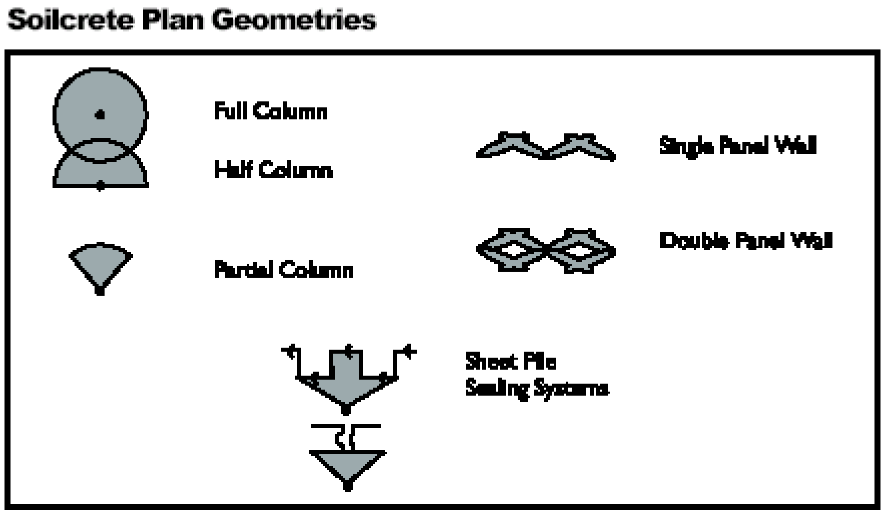


Figure 8.6 Jet Grouting Geometries (from Hayward Baker)

Table 8.1 Summary of Jet Grout Column Properties

Soil Type	Column Diam. Single Rod, m	Column Diam. Triple Rod, m	Compressive Strength, kPa	Coeff. of Permeability, m/s
Clean Sand & gravels	0.9 – 1	0.8 - 1	6900 - 17,000	10^{-7} to 10^{-9}
Silty Sand	0.8 - 0.9	1.4 - 1.6	4800 - 10,000	10^{-7} to 10^{-10}
Clayey Silts	0.4 - 0.5	0.8 - 1.0	4000 - 18,000	10^{-7} to 10^{-10}
Clays	-	-	500 - 8,000	10^{-7} to 10^{-10}

Costs (FHWA, 1998)

The cost of jet grouting is highly dependant upon complexity of the problem, soil type, and depth of treatment. Costs at the complex Boston Artery project were approximately $200/m^3 treated. Mobilization and demobilization costs range from about $25,000 to $50,000. Underpinning and excavation support grouting cost estimates are $95-$550 per meter for 1.0 m diameter columns. Similar dimension columns for seepage control have cost ranges of $30-$115 (FHWA, 1998).

Theoretical Aspects – Practical Implications

Shibazaki (2003) indicates there exists a critical waterjet pressure required for erosion. Although erosion occurs below these critical pressures, lower pressures are not economical. His experimental evidence suggests critical pressures of 80 kPa for sandy soils and 150 kPa for clayey soils.

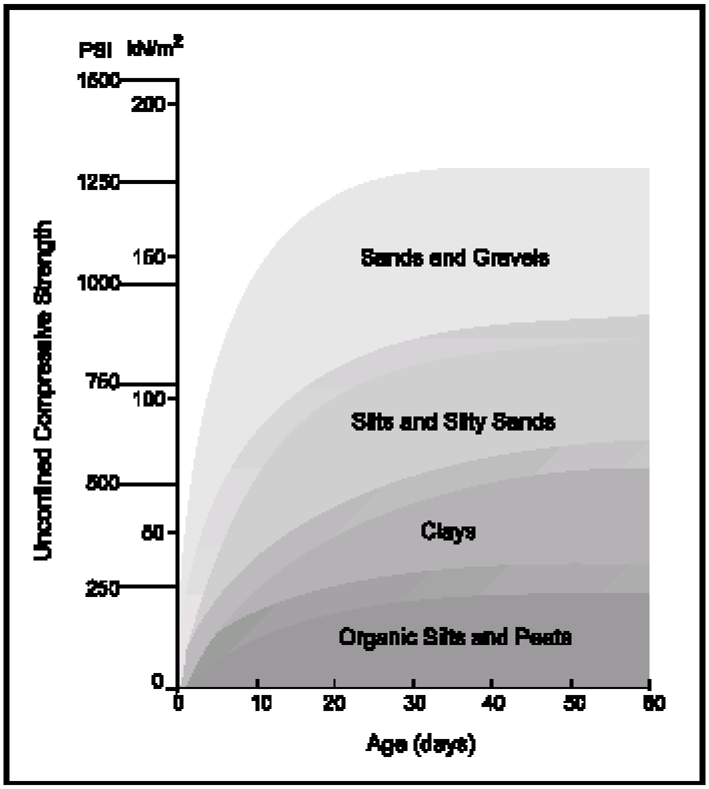


Figure 8.7 Typical Jet Grout Strengths (from Hayward Baker)

The grout column radius experimentally reported by Shibazaki (2003) is a function of these variables:

L_m = improved column radius, m

K = soil type coefficient, 31.5 for sandy soils

P = jetting pressure, Mpa

Q = jetting flow rate, m3/min

N = number of revolutions at given depth

V = velocity of nozzle rotation, m/s = (Diam × π × rotating speed, rpm)/60

D = external diameter of a monitor

α = 1.003 β = 1.186 γ = 0.135 δ = 0.198

where: $L = (KP^{\alpha}Q^{\beta}N^{\gamma}) \div V^{\delta}$

Thus, for a jet grouting column using these conditions;

P = 40 Mpa,

Q = 70 ltr/min -> 0.0012 m^3/sec,

N = 2,

D_m = 90 mm diameter tool = 0.09m

R_s = 5 rpm

Vn = 0.09 x π x 5rpm/60 = 0.0236 m/sec

Then,

$$Lm = 31.5 \times 40^{1.003} \times 0.0012^{1.186} \times 2^{0.135}/0.0236^{0.198} = 1.009m$$

This equation shows that erosion efficiency is proportional to jetting pressure, flow rate, number of revolutions at given depth, and inversely proportional to the rpm's of the nozzle. His experiments suggest that less than 5 repetitions at a given depth may affect efficiency and the diameter of the grout column. Because the number of revolutions at a given depth improves grouting, he recommends jet grouting in successive lifts, rather than, steady lifting (withdrawal) of the jet.

REFERENCES

Brill, G.T., Burke,G.K., and Ringen, A.R. (2003) "A Ten-Year Perspective of Jet Grouting: Advancements in Application and Technology" ASCE Grouting and Ground Treatment Proc 3rd International Conference, Geotech. Spec. Publ. 120 pp 218-235.

FHWA (1998). ***Ground Improvement Technical Summaries, Final Draft,*** FHWA Demonstration Project No. 116, Contract No. DTFH61 -93-R-00 145, Elias, V., Welsh, J., Warren J. and Lukas R.

Hayward Baker www.haywardbaker.com *Jet Grouting.*

Miki, G. (1984) "Japanese Experience on Sub-Soils reinforcement by Jet Grouting," Lecture Notes for Address Proc. Conf. In-Situ Soil and Rock Reinforcement, Paris 9-11 October, Reported in Moseley, 1993).

Moseley, M.P. (1993) *Ground Improvement*, Blackie Academis & Professional Publ CRC Press, Boca Raton, FL.

Shibazaki, M (2003) "State of Practice of Jet Grouting," ASCE Grouting and Ground Treatment Proc 3rd International Conference, Geotech. Spec. Publ. 120 pp 198-217.

Weaver, K. (1991) *Dam Foundation Grouting* ASCE Publ. Ruston, VA.

Chapter 9

DEEP SOIL MIXING

PURPOSE

The purpose of deep soil mixing is the in situ improvement of soils at depth without excavation by introducing reagents (lime, lime-cement, and cement). The desired benefits being: for ground-water cutoff, excavation support, soil stabilization, foundation support, and fixation of contaminated soils. Additional information can be obtained from;

http://tc17.poly.edu/Deep_Soil_Mixing.htm and www.swedgeo.se/sd.htm

BACKGROUND

Deep stabilization of soft soils with lime-cement columns has been used in Sweden since 1975 for reinforcement of the soil. With this technique the soft soil is mixed in situ with a stabilizing material creating a column with a diameter of 0.5 to 1.2 meter down to a depth of normally 15 to 25 meters. During the last 4 to 5 years, the use of the method has increased substantially and the method has become the most frequently used reinforcement method in infrastructure projects. About 3 to 4 millions linear metres of lime-cement columns have been installed annually the last years.

Another technique that has been used during the last years is mass stabilization. By this technique the upper part of the soft soil is mixed horizontally as well as vertically. The methods are primarily used for reduction of settlements and for improvement of stability, mainly in infrastructure projects, such as roads and railways on soft soil deposits. It is also used for foundation of smaller buildings and bridges as well as for stabilization of excavations and natural slopes. The method is mainly used in soft clays but also in organic clays and clayey silts.

The experience of these methods is very favorable from a technical aspect and from an economic point of view as well. Considerable cost reductions can be made compared with alternative methods. There is also a reduction in construction time (the time-to-market) for infrastructure projects compared with other methods. Furthermore, less maintenance is required. A final and important advantage is the fact, that the method is environmentally friendly (Swedgeo).

Figures 9.1 to 9.3 illustrate examples of the equipment and applications.

History (Yang, 1997)

Various methods of soil mixing, mechanical, hydraulic, with and without air, and combinations of both types have been used widely in Japan for about 20 years and more recently have gained wide acceptance in the United States. The soil mixing, ground modification technique, has been used for many diverse applications including building and bridge foundations, retaining structures, liquefaction mitigation, temporary support of excavation and water control. Names such as Jet Grouting, Soil Mixing, Cement Deep Mixing (CDM), Soil Mixed Wall (SMW), Geo-Jet, Deep Soil Mixing, (DSM), Hydra-Mech, Dry Jet Mixing (DJM), and Lime

Figure 9.1 Illustration of Deep Mixing Equipment (from Swedgeo)

Figure 9.2 Photo of Soil Mix Columns (from Swedgeo)

Columns are known to many. Each of these methods has the same basic root, finding the most efficient and economical method to mix cement (or in some cases fly ash or lime) with soil and cause the properties of the soil to become more like the properties of a soft rock.

At the present time, the total volume of soil mixing work performed annually in Japan is about 5,000,000 million cubic meters. This includes CDM, SMW, DJM and Jet Grouting. Total Yen (Dollar) volume annually is on the order of $2 Billion performed by over 500 rigs in all categories operating throughout the country. In contrast, in the United States, currently there are about 10 traditional soil mixing rigs operating, plus about another 10 jet grouting rigs, and total of soil treated in any one year has not exceeded approximately 30,000 cubic meters. Japan has a population of about 1/2 that of the United States and a land area smaller than the state of California.

Therefore, their need to utilize all available space has made reclamation of soft soils along their coastline critical to providing the needs of their population. Much of their soil mixing has been to treat soft bay muds in coastal areas, developing strengths of 5-20 kg/cm2 (75-300 psi).

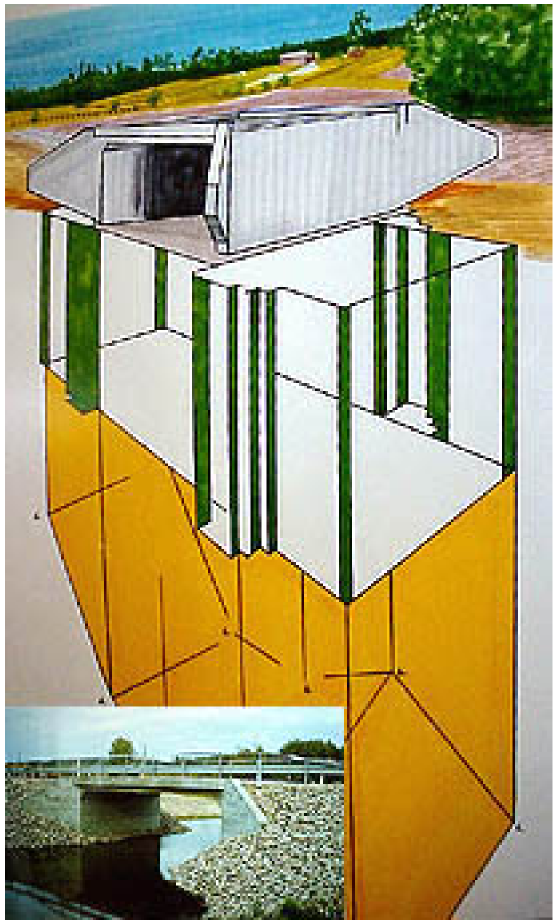


Figure 9.3 Deep Mixing at Bridge Abutment (from Swedgeo)

At about the same time as the use of soil mixing was expanding in Japan (mid 1970'S), independent progress was being made in Scandanavia with a lime column technique for the stabilization and reinforcement of very soft, cohesive soils. This technology has evolved in Sweden and Finland to the present time where production of what is now lime-cement columns ranges between 3 and 4 million lineal meters per year. This production is mainly for reduction of settlements and improvement of stability for the construction of new roads and railroads. Almost all this production is constructed using dry reagents that are introduced by compressed air and mixed mechanically with the soft soils.

APPLICATIONS

Figure 9.4 illustrates various applications of deep soil mixing (CDM, 1994).

Classification (FHWA, 1998)

Deep soil stabilization is performed worldwide under a variety of names. Consequently, the following classification has been suggested:

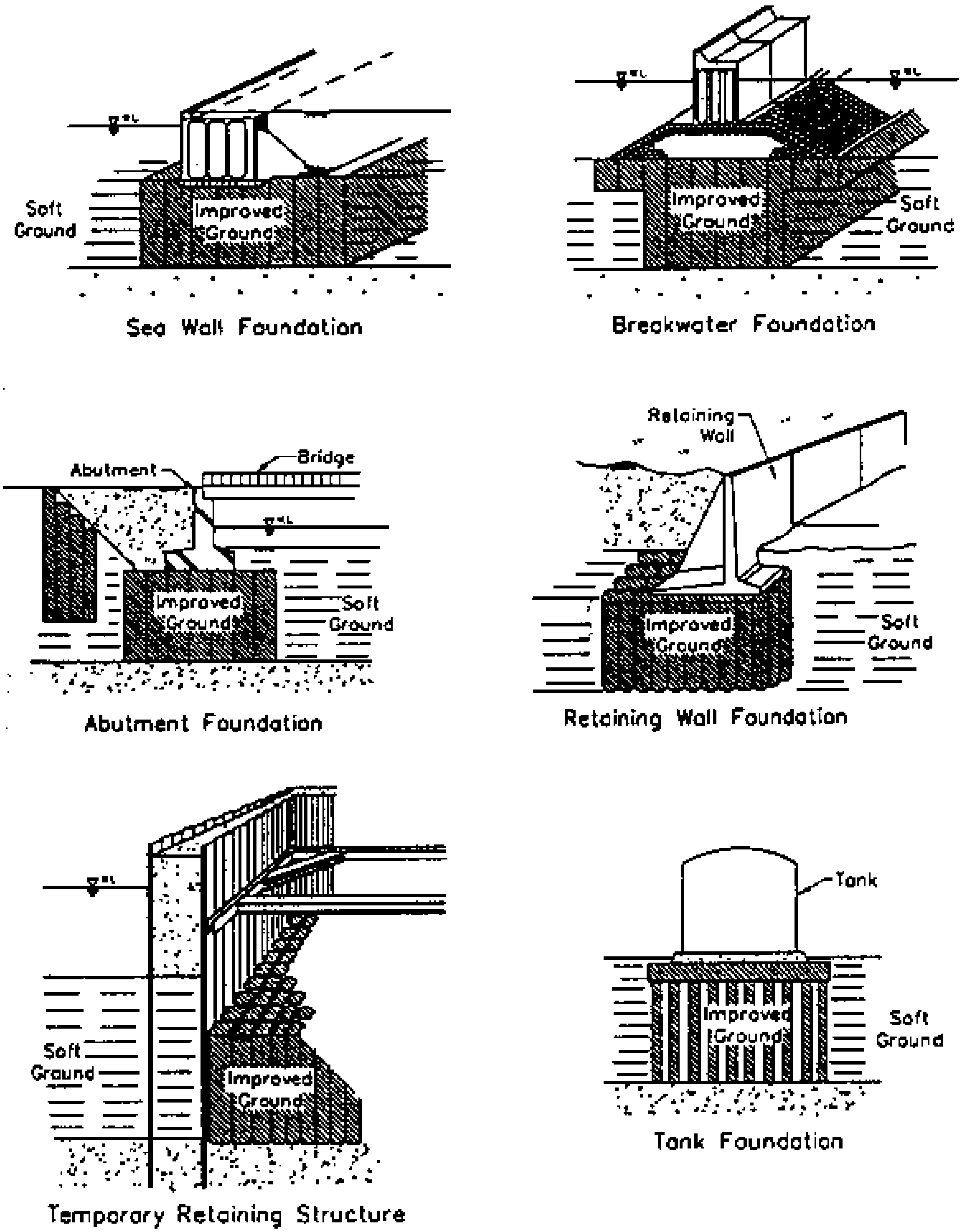


Figure 9.4 Examples of Deep Soil Mixing Applications (from CDM, 1994)

1. Method of additive injection: Wet (W) or Dry (D)
2. Method of mixing: Rotary (R). Jet (J)
3. Location of mixing action: Near drilling tool (E), along shaft (S)

From this classification two distinct groups result:

(1) Deep Soil Mixing (WRS,WRE,WJE) - wet block or wall techniques using primarily cement based grouts developed for large scale foundation improvement and soil containment.

(2) Lime Columns (DRE) - dry auger column technique for lime or lime cement mixing developed for soil stabilization and reinforcement of cohesive soils.

Applications for Deep Soil Mixing

1. Foundation Improvement - Used to provide stable bearing capacity for structural loads and reduce settlements
2. Liquefaction Mitigation - Improves shear strength to contain liquefiable deposits
3. Excavation support walls - Support walls using H piles in auger-holes and soil-cement functions as lagging. May be anchored
4. Cut-off wall - Containment barrier permeability 10^{-4} - 10^{-7} cm/sec.
5. Hazardous waste containment - solidify or bind waste

Applications for Lime and Lime/Cement Columns

1. Settlement control- rationale of stone columns. Lime only as wick drains
2. Improved stability - Increase of mass shear strength for stability of embankments, slopes, trenches, and deep cuts
3. Increase sheet-pile stability - Increase passive resistance at toe, or reduce active pressure behind wall by increased strength.

Advantages and Disadvantages of Deep Soil Mixing (FHWA, 1998)

Advantages

1. Improvement up to 30 m. in a variety of soils from soft ground to weathered rock, but most likely in soft clays.
2. Avoids costly dewatering
3. More economical that removal and replacement
4. No noise or vibration problems
5. High production rate when using multi-axis augers

Figure 9.5 presents typical grid patterns for deep soil mixing.

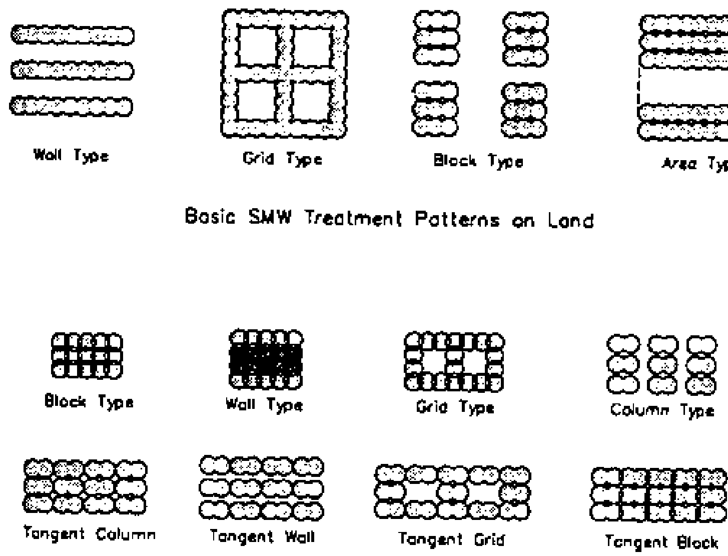


Figure 9.5 Basic Deep Mixing Treatment Patterns

Disadvantages

1. High mobilization costs plus cost of batch plants
2. Needs a thorough site/soils investigation program. Lab tests need to determine soil-reagent compatibility. Strength time dependent up to 6 months
3. Rigs require substantial headroom
4. Although waste is generally less than other methods, deep soil mixing produces 30 - 50% of spoil soil volume
5. Lack of well-developed design and analysis models limits acceptance.

Advantages and Disadvantages of Lime-Cement Columns

Advantages

1. Provides an economical alternative to wick drains, drilled shafts, lightweight fills, or surcharging.

Disadvantage

1. In low pH (acidic) soils the full strength of the column may not be fully developed. Strength development is time dependent and not fully reached for several months; consequently reduced (28 day) strengths may be used for design. The strength of the stabilized soil should be at least 3-5 times stronger than insitu soil to be economically viable.

Site Investigation: (FHWA,1998)

Deep soil mixing requires the use of very large equipment, therefore restricted site access, inadequate working areas or the presence of overhead utilities preclude this technology. The site investigation should include;

1. soil type. the existence of organics or acidic soils, or gravel obstructions
2. in situ water content
3. engineering property determinations, strength, compressibility, index properties and classification
4. pH
5. chemical and mineralogical properties to assess pozzolanic reactivity
6. ground water levels

Table 9.1 summarizes the range of anticipated engineering properties that can be achieved by deep soil mixing (Bruce, 1998).

Table 9.1 Typical Properties Lime-Cement

Property	Range
U/C strength	0.2-5.0 MPa-granular soil 0.2-3.0 MPa cohesive
Permeability	10^{-6} - 10^{-9} m/s
Modulus	350 to 1000 q_u - lab samples 150 - 500 q_u -field samples
Tensile strength	8 – 10% q_u

Certain soil chemistry factors significantly impede soil stabilization with cement as summarized in Table 9.2 (FHWA,1998).

Table 9.2 Favorable Soil Chemistry Factors

Property	Favorable Soil Chemistry
Natural water content, w	Should be < 200%
Organic content	Should be < 6% (wet method)
Loss on ignition	Should be < 10%
Humus content	Should be < 0.8%
Conductivity	Should be > 0.4 mS/cm
pH	Should be > 5

Preliminary lab testing: (1) multiple water/reagent ratios, (2) multiple percentages, (3) multiple mix times.

TREATED SOIL CHARACTERISTICS (Nicholson, 1998)

As stated, the intent of most soil mixing is to modify the soil so that it's properties become similar to that of a soft rock such as a clay shale or lightly cemented sandstone. The modulus of elasticity and unconfined compressive strengths are typically 1/5 to 1/10 that of normal concrete. Almost all soil types are amenable to treatment; however, soils containing more than 10% peat must be tested thoroughly prior to treatment. Mixing of soft, clay soils must be carefully controlled to avoid significant pockets of untreated soils. However, there are methods readily available to insure competent mixing and methods of testing to insure that adequate mixing and treatment has been achieved.

Soil mixing is also commonly used as a stabilization or in-situ fixation method for containing hazardous wastes and sludges. Containment walls can be constructed with permeability of approximately 5x10-7 cm/sec, similar to that achieved by most slurry wall techniques.

See Table 9.3 for typical strength and permeability characteristics of treated soils.

Table 9.3 Typical Strength and Permeability Characteristics of Deep Mix Stabilized Soils

Soil Type	Cement Usage	UCS	Permeability
Sludge	240 to 400 kg/m^3 (400 to 700 lbs/cy)	70-350 kPa (10-50 psi)	1×10^{-6} cm/sec
Organic silts and clays	150 to 260 kg/m^3 (260 to 450 lbs/cy)	350-1400 kPa (50-200 psi)	5×10^{-7} cm/sec
Cohesive silts	120 to 240 kg/m^3 (200 to 400 lbs/cy)	700-2100 kPa (100-300 psi)	5×10^{-7} cm/sec
Silty sands and sands	120 to 240 kg/m^3 (200 to 400 lbs/cy)	1400-3500 kPa (200-500 psi)	5×10^{-6} cm/sec
Sands and gravels	120 to 240 kg/m^3 (200 to 400 lbs/cy)	3000-7000 kPa (400-1000 psi)	1×10^{-5} cm/sec

Deep Soil Mixing – Materials (FHWA,1998)

Deep soil mixing was initially developed using cementing as the mail stabilizer and later lime column technology was developed using lime. Currently, a lime-cement mixture with lime forming 15-40% is most commonly used. In the wet mixing process cement is introduced as a slurry with bentonite added to prevent the cement from settling out of solution.

In lime columns dry unslaked lime is used. Consequently, heat is generated. Gypsum may be added as an accelerant for additional strength.

ENGINEERING PROPERTIES OF SOIL-CEMENT

The major factors that influence the engineering properties of soil-cement include soil type, amount of cement or other hardening reagents used, water cement ratio of grout, degree of soil-cement mixing, curing environment, and age. Either sea water or fresh water can be used for deep mixing. Sea water has been used for most of the marine construction work since there is no difference in strength affected by the use of either sea water or fresh water (CDM 1994). Fresh water was used for most of the construction on land. Considering the application of the soil-cement wall for excavation support, groundwater control, and soil stabilization, the engineering properties of major concern are strength, permeability, compressibility, and modulus of elasticity.

Strength

The strength of soil-cement can be obtained in a laboratory by performing unconfined compressive strength tests, triaxal compression tests, direct shear tests, and tensile tests. The test samples include laboratory samples, field wet samples, and core samples prepared before, during, and after construction, respectively. The most common type of test is the unconfined compressive strength test and its results are used for design, construction quality control and quality assurance.

Soil type is the most dominant factor that influences the strength of soil-cement. The same treatment used in different soils produces results with a wide variation. The effect is attributed to the adsorption and pozzolanic reaction in the various soils as well as the reaction of the hardening reagent itself. It limits the strength of the soil-cement in a certain range beyond which the design becomes not cost efficient or even impractical. Figure 9.6 shows the unconfined compressive strength of soil-cement walls installed in clayey soil, sandy soil, and gravelly soil obtained from dozens of soil-cement well projects. The increased dosage of cement increased the strength of the soil-cement for each type of soil. The increase of strength in cohesive soils due to increased amounts of cement is minor in comparison with those in sandy and gravelly soils.

The correlation between unconfined compressive strength and shear strength obtained from direct shear tests in represented by the following equation (Saito et al. 1980).

$$\tau_0 = 0.53 + 0.37\, q_u - 0.0014\, q_u^2 \;(q_u \leq 60 \text{ kg/cm}^2)$$

where

t_0 : 28-day shear strength (kg/cm^2) obtained by direct shear test with zero normal stress

q_u : 28-day unconfined compressive strength (kg/cm^2).

The q_u to τ_0 ratio is approximately 2 when q_u is less than 10 kg/cm^2 (142 psi). This ratio reduces gradually as q_u increases. The tensile strength of soil-cement is measured by direct uniaxial tensile tests or splitting tensile strength tests. The latter provides lower or conservative tensile strength. For soil-cement with unconfined compressive strength less than 60 kg/cm^2 (852 psi), the ensile strength obtained by splitting tensile strength tests on laboratory samples varies from 8 to 14 percent of the unconfined compressive strength. Splitting tensile strength testing of field samples indicated similar results (Terashi et al. 1980, Nakajima et al. 1981, and CDM 1994).

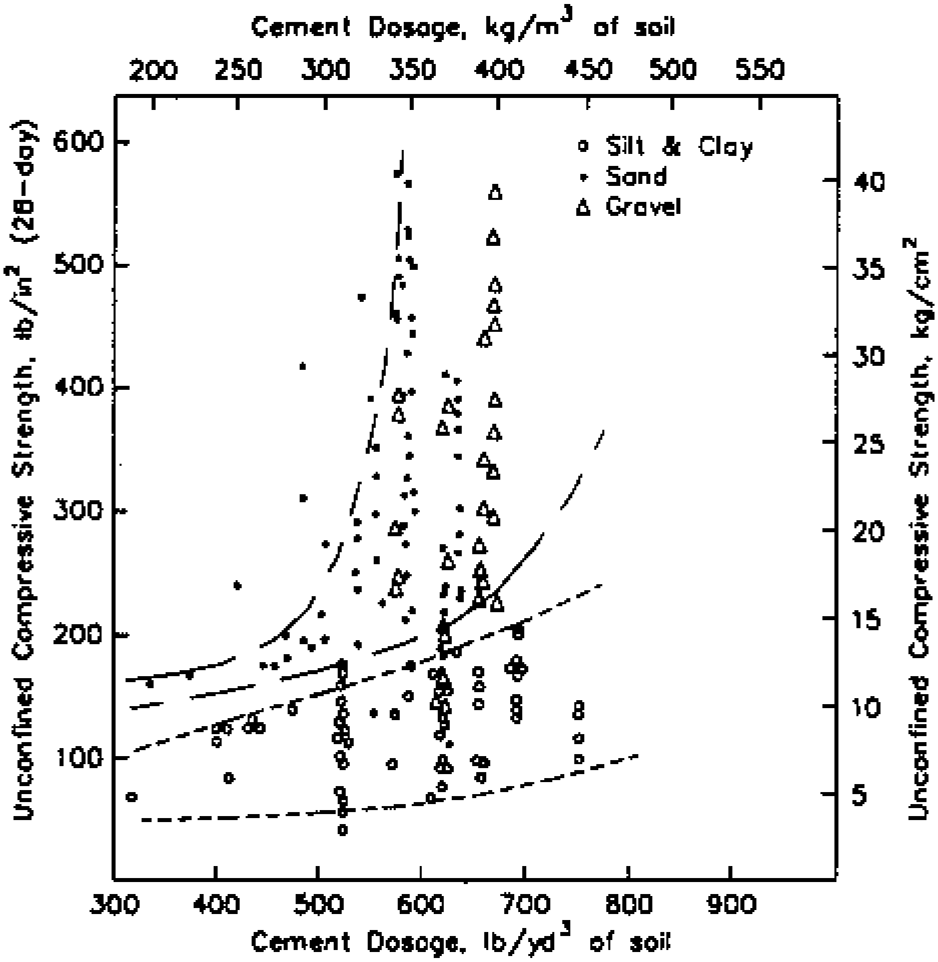


Figure 9.6 Strength of Soil-Cement (from Taki and Yang, 1991)

Coefficient of Permeability

The coefficient of permeability of soil-cement is affected by the soil type, the amount of cement and bentonite used, the water cement ratio, the grout injection ratio and age. Cement and bentonite dosage is used to control the permeability of soil-cement walls. The coefficient of permeability of the soil-cement ranges from 10^{-7} and 10^{-9} m/sec based on laboratory testing of field wet samples obtained during construction (Yang et al. 1993). For use as excavation support and groundwater control, soil-cement walls with coefficient of permeability in the order of 10^{-5} m/sec are considered satisfactory. A coefficient of permeability of 10^{-5} m/s or less is usually required for pollution control or permanent seepage control in dams, dikes, or dry dock projects. In cases where a permeability of lower than 10^{-9} m/s is required, bentonite or clay-bentonite slurries have to be used for mixing with in situ soil (Yang 1995).

Modulus of Elasticity and Poisson's Ratio

The modulus of elasticity (E_{50}) of soil-cement is proportional to the unconfined compressive strength (q_u) with a ration of 350 to 1000 (Saito et al. 1980) and is affected by the same factors that influence the strength of soil-cement. For cohesive soils with sand content less than 10 to 15 percent, the E_{50} to q_u ratio is between 400 to 600. The ratio is obtained from the stress-strain curve of soil-cement samples. A recent study indicates that this ratio is dependent on the methods used to measure strain during load tests. Local and sensitive measurements of axial strains using local deformation transducers (LDT) were found imperative for accurate evaluation of the stiffness of soil-cement at small strains which are expected to occur in the field of soil-cement masses at working loads (Tatsuoka et al. 1996).

The static Poisson's ratio is approximately 0.5 if the in situ soil-cement is loaded under undrained conditions and ranges between 0.3 to 0.45 under other loading conditions (CDM 1994).

DESIGN CONCEPTS

For excavation support, design is similar to flexible walls. The pattern is usually overlapping single or double row columns, Reinforcement members (H-piles) are inserted to resist moments and deflections.

For excavation support, the analysis consists of determining the maximum bending moment and shear using the bending rigidity of the reinforcing member, and by determining the bending stress and shear stress using section properties of a unit width of the reinforcing member. An empirical criterion for spacing reinforcement members to avoid a bending failure of the soil-cement is presented in Figure 9.7. The spacing is given as:

$$L_2 \leq D + h - 2e$$

where

L_2 = H-pile flange to flange,

D = diam of soil-cement column, and

e = difference between eccentricities of H-pile and Diameter.

For liquefaction mitigation, a lattice pattern is typically used, as illustrated in Figure 9.8.

For stabilization as structural support a continuous treated soil mass is used as shown in Figure 9.5. The massive block is consider as rigid structural member. Routine checks of external stability; i.e., sliding, overturning, and bearing cap using active and passive forces of untreated soil are used.

Field strengths are about 0.5 to 0.2 those of the lab.

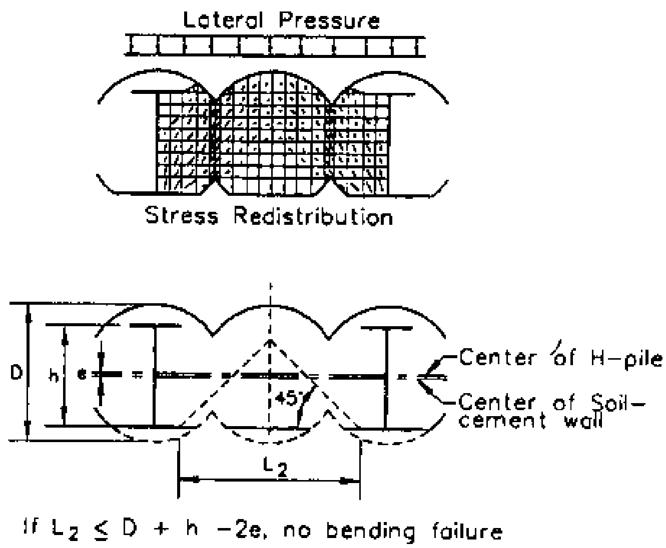


Figure 9.7 Stress-Analysis of Soil –Cement Wall (from Taki and Yang, 1991)

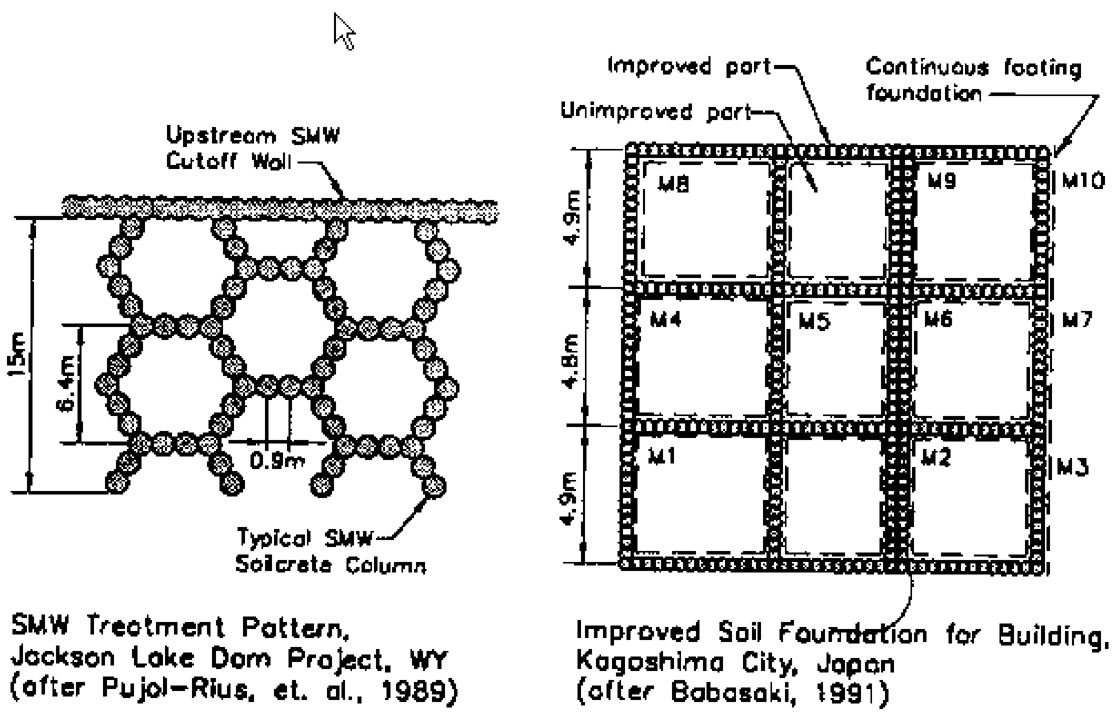


Figure 9.8 Examples of Deep Mixing for Liquefaction Mitigation (from Yang, 1998)

LIME COLUMNS

Lime columns are typically used for settlement, stabilization, and slope stability; in ratios of 75:25 to 85:15 PC:CaO. For design 100 to 200 kPa is the max. shear strength. For slope stability applications a recommendation is to use an average shear strength $\tau = \tau_{column}\ a + C_u\ (1-a)$, a = area replacement ratio = ratio of column area/(horizontal distance)^2 . Typical lime column properties are listed in Table 9.4

Table 9.4 Typical Properties of Lime Columns

Property	Range
Shear strength, C_u	10 -50 times C_u
Modulus	50 to 200 C_u
Fail strain	< 2%
Permeability (lime/cement) (lime)	about = in situ soil 100 to 1000 times in situ soil

Ultimate bearing capacity of a single lime column is governed by (1) strength of surrounding soft clay, or (2) column strength: (ASCE,1996)

1. $Q_{ult,soil} = (\pi d H_{col} + 2.25\pi d^2)C_u$, where d = diam. and H = ht. of column This assumes side shear is = C_u soil and tip = $9C_u$, if $C_u > 30$ kPa use 0.5 C_u
2. $Q_{ult,col} = A_{col}(3.5C_{col} + 3\sigma_h)$ where C_{col} = cohesion of soil-lime column, $\sigma_h = (\sigma_{v\ total} + 5\ C_{u\ clay})$, this is based upon $\phi_{u,\ col} = 30°$; giving $K_p = 3.0$
3. Ultimate capacity of lime column group depends upon strength of untreated soil and lime columns

CONSTRUCTION METHODS AND CONSTRUCTION EQUIPMENT

Mechanical soil mixing is typically performed using single or multiple shafts of augers and mixing paddles. The auger is slowly rotated into the ground, typically 10-20 rpm, and advanced at 0.5-1.5 meters per minute. As the auger advances, cement slurry is pumped through the hollow stem of the shaft(s) feeding out at the tip of the auger. Mixing paddles are arrayed along the shaft above the auger to provide mixing and blending of the slurry and soil. (See Figure 9.9 for typical multiple stem auger and mixing paddles). The slurry helps to lubricate the tool and assists in the breaking up of the soil into smaller pieces. Since fluid volume is being introduced into the ground, spoils must come to the surface. These spoils are a combination of the cement slurry and soil particles, typically with a similar cement content as what remains in the ground. After final depth is reached, the tools remain on the bottom of the hole, rotating for about 0.5 to 2 minutes for complete mixing. At this point, the tools are raised while continuing to pump slurry

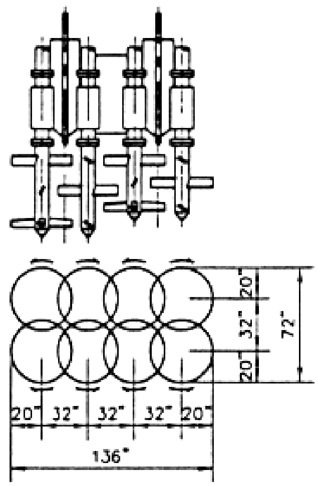
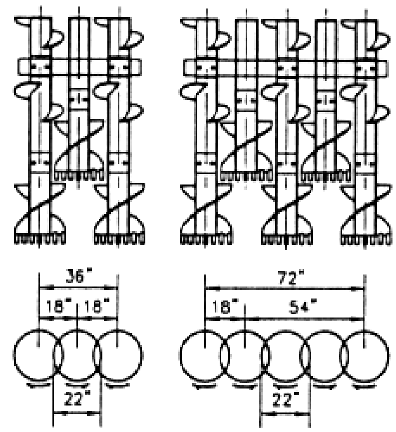
Title	CDM Cement Deep Mixing	SMW Soil Mix Wall
Sketches of Representative Mixing Mechanisms	 ø = 39" to 63" available 1 ft = 0.305 m	 ø = 22" to 40" available 1 ft = 0.305 m
Descriptions	Rotation of multiple axis shafts create relative movement and shear in soil for soil-reagent mixing.	Uses multiple auger, paddle shafts rotating in alternating directions to mix in situ soil with cement grout or other reagents to form continuous soil-cement walls.
Number of Mixing Shafts	2,4,6,8 shafts.	1, 2, 3, & 5 shafts.
Major Reagents	Cement grout, lime or other cementitious slurry.	Cement grout, bentonite slurry, clay slurry, or other stabilizing reagent slurries.
Applicable Subsurface Soils	Very soft silt and clay or very loose sandy soils usually undersea.	Soft to hard silt and clay, loose to very dense sand, gravel, an cobble soils. Cobble and boulder soil and bedrock with predrilling.
Major Applications	Large scale soil stabilization of sea floor for offshore or waterfront development.	Continuous walls for excavation support and groundwater control; Column blocks, lattice, or areal patterns for stabilization.
Remarks	Developed by Port and Harbor Research Institute.	Developed by Seiko Kogyo, Co. Ltd.

Figure 9.9 Deep Mixing Equipment Features (from Taki and Yang, 1990)

at a reduced rate. Withdrawal is typically at twice the speed of penetration, 1-3 meters per minute.

Other methods of mixing cement with soil consist of jet grouting. Here, high-pressure cement slurry (4-7000 psi) is pumped through horizontal ports in a drill string above the drill bit. The high velocity and pressure of the cement jets cuts and mixes the soil in situ. This is termed single fluid jet grouting. In double fluid jet grouting, a shroud of compressed air (10-15-bar pressure) is pumped to surround the slurry jet thus enhancing the penetrating ability of the jet. In triple fluid jet grouting, the cement is pumped at low pressure at the bottom of the hole while high pressure water, surrounded by a shroud of compressed air, cuts and removes the soil during the withdrawal of the tools.

Other methods of introducing and mixing cement with soil involve such methods as Hydra-Mech, utilizing both hydraulic (jet) and mechanical energy to cut and mix the soil and cement. A different method not utilizing slurry is the DJM (Dry Jet Mixing) or Lime Column method. Here, compressed air carries lime and/or cement powder to the bottom of the hole where mixing paddles blend the dry reagent with the soil. This method can only be used in high moisture content, soft soils.

Treatment of the soil can be done to a replacement ratio of 100% wherein all the soil inside a particular block is treated to a specified strength by mixing with cement. Other patterns, as shown in Figure 9.5, can be employed to achieve the desired result. Recently a prototype retaining wall was constructed at the National Geotechnical Experimentation Test Site at Texas A & M University that employed a less than 100% treatment ratio to achieve a composite behavior of a block of soil. In this test, a replacement ratio of approximately 35% cement treated to native soil ratio was used to force a composite action of the soil cement columns and native soil. This gravity wall, installed in May 1998, 11 meters high, containing no steel reinforcing, has performed well to date with total movements of 30 mm.

QUALITY CONTROL AND TESTING

Since the aggregate being used in producing the engineered "low strength" concrete in situ is the native soils, pre-construction soil borings, testing of the mix design with the in-situ soils is a must. One to two cubic feet of the soils is sufficient to run the required laboratory, pre-production tests on the soil cement mix. Various water cement ratios are considered, usually between 1:1 and 1.5:1 (by weight). The amount of cement, again by weight, is typically 5-15% of the weight of the soil to be treated.

Proper injection of slurry, mixing and blending of the cement slurry and soil is verified by several means. Initially, during installation, wet grab samples are taken from different elevations in the mixed columns after the tools are withdrawn. Remote closing tubes are inserted, filled with the wet, mixed soil and slurry, a closure lid secured and the sample brought to the surface. The slurry is poured into cylinders for later laboratory testing. In addition, core sampling of the completed columns may be performed. It is wise to wait at least until 28 days after installation to perform coring, and then only with triple tube coring equipment, as the sample may be difficult to retrieve intact because of it's low strength.

COST (http://www.gnet.org/archive/4591.html)

Updated 1996 costs are estimated at $120 - 175/cy or less.

REFERENCES

ASCE (1996) Soft Ground Improvement in Lowland and Other Environments editors: D.T. Bergado, A.S. Balasubramaniam, N. Miura, and L.R. Anderson pp. 278.

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

CDM (Cement Deep Mixing) (1994) CDM Design and Construction Manual, CDM Construction Institute (reported by Yang, 1997).

FHWA (1999) Ground Improvement Technical Summaries Demonstration Project 116, Vol. 1 and 2, Publication No. FHWA-SA-98-086, Authors: Elais, V., Welsh, J., Warren, J., and Lukas, R.

Nicholson, Peter J. (1998) Abstract Cement Soil Mixing in Soft Ground http://cigmat.cive.uh.edu/content/conf_exhib/99_present/13.htm

Swedgeo (2004) Swedish Deep Stabilization Research Centre, www.swedgeo.se/sd

Taki, O. and Yang, D.S. (1991) "Soil Cement Mixed Wall Technique" ASCE Special Conference, Denver, CO (reported in Yang, 1997)

Tatsuoka, F., Kohata, Y., Uchida, K., and Imai, K. (1996) "Deformation and Strength Characteristics of Cement Treated Soils in the Trans-Tokyo Bay Highway Project" Grouting and Deep Mixing, IS-Tokyo '96, The 2nd International Conference on Ground Improvement Geosystems, Tokyo, pp. 453-459

Terashi, M., Tanaka, H., Mitsumoto, T., Shindome, Y., and Honma, S. (1980) "Fundamental Properties of Lime and Cement Treated Soils" (2nd Report) Report of the Port and Harbor Research Institute, Vol. 19, No. 1.

Yang, David S. (1997). Ground Improvement, Ground Reinforcement, Ground Treatment: Developments 1987-1997, Edited by Schaefer, V.R., Associate Editors: Abramson, L.W., Drumheller, J.C., Hussin, J.D., and Sharp, K.D., ASCE Geotechnical Special Publication No. 69, pp. 130-150.

Yang, D.S., Luscher, U., Kimoto, I., and Takeshima, S. (1993) SMW Wall for Seepage Control in Levee Reconstruction" Third International Conference on Case Histories in Geotechnical Engineering, St. Louis, MO, pp. 487-492.

Chapter 10

MSE (Reinforced Earth) RETAINING WALLS

HISTORY

The concept of soil reinforcement is not new. In nature, birds build nests out of mud reinforced with twigs, and beavers build earth dams of mud strengthened by sticks. Adobe bricks of clay reinforced with straw are mentioned in the Old Testament (Exodus 5:6-9).

The modern concept of soil reinforcement for retaining wall construction was pioneered by Henri Vidal, who in patented "Terre Armee" or "Reinforced Earth" as French patent No. 929421, March 1963. In 1972, the first wall using this technology was constructed on California State Highway 39 near Los Angels. Since then, more than 23,000 Reinforced Earth structures representing over 70 million m^2 (750 million ft^2) of wall facing have been completed in 37 countries. More than 8,000 walls have been built in the United States since 1972. The highest wall constructed in the United States was on the order of 30 meters (98 feet). (FHWA ,1998)

Currently, most patents involving reinforced – soil have expired, and the remaining patents in force cover connections between reinforcement and the facing. The primary differences being the reinforcement as shown in Table 10.1 (FHWA,1998)

Table 10.1 Summary of Reinforcement and Panel Details for MSE Walls

System Name	Reinforcement	Facing Panel
Reinforced Earth	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	Rectangular grid of W11 or W20 plain steel bars, 610 x 150 mm grid. Each mesh may have 4 to 6 longitudinal bars	Hexagonal and square precast concrete 1.5 m x 1.5 m x 140mm thick panels. ½ panels used top and bottom
Hilfiker Welded Wire Wall	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).
MESA Tensar Earth Technologies, Inc.	HDPE Geogrid	MESA HP (high performance), or Standard units (203 mm high by 457 mm long face, 275 mm nominal depth). (dry cast concrete)

Figure 10.1 presents a schematic of a Reinforced Earth® wall and its components. As shown the wall systems consists of:

a. A leveling pad (footing) upon which the wall facing panels are founded.

b. Facing used to prevent the soil from raveling out between the rows of reinforcement. Common facings include precast concrete panels, dry cast modular blocks, metal sheets and plates, gabions, welded wire mesh, and wrapped sheets of geosynthetics. The facing also plays a minor structural role in the stability of the structure

c. Reinforcement encompasses man-made elements incorporated in the soil to improve its behavior via stress transfer. Examples of inclusions are steel strips, geotextile sheets, and steel or polymeric grids.

d. Reinforced backfill is the select fill material located between the mechanically stabilized soil mass and the natural soil. It is the fill material in which the reinforcements are placed.

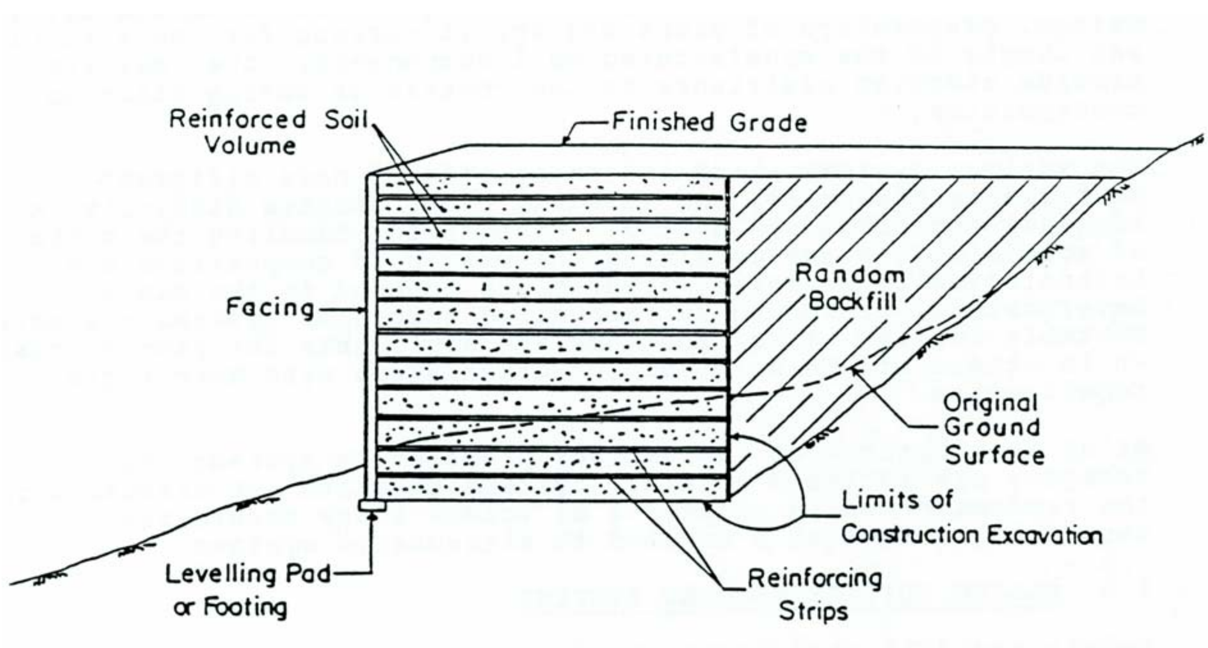


Figure 10.1 Schematic of MSE Wall (from FHWA, 1990)

Figures 10.2 – 10.4 illustrate examples of wall systems listed in Table 10.1.

APPLICATIONS

MSE systems are extremely versatile and are cost-effective where most concrete gravity or cantilever walls have been traditionally used. These include; retaining walls, bridge abutments, and seawalls. Prefabrication and element construction permit the use of small equipment, and semiskilled labor for construction. They are readily adapted steep-sided terrain, areas of poor foundation soils, and earthquake prone regions.

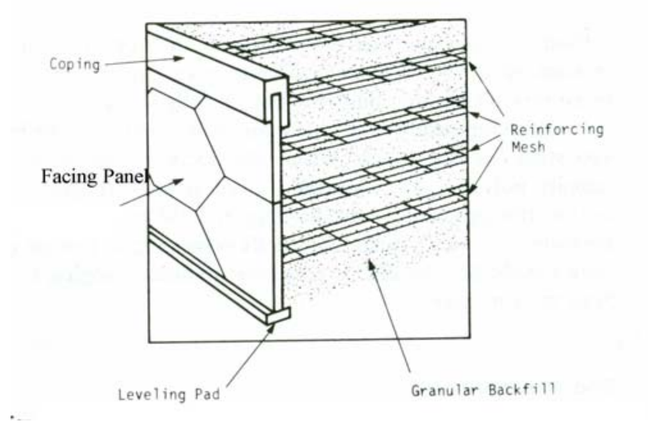


Figure 10.2 Schematic of VSL MSE Wall (NCHRP,1987)

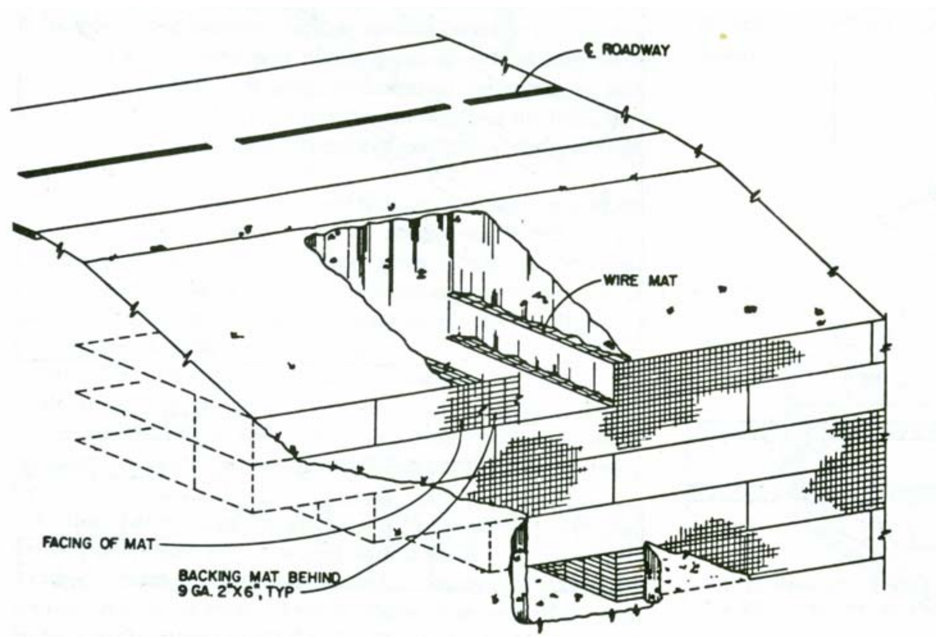


Figure 10.3 Schematic Example of Welded Wire MSE Wall (NCHRP,1987)

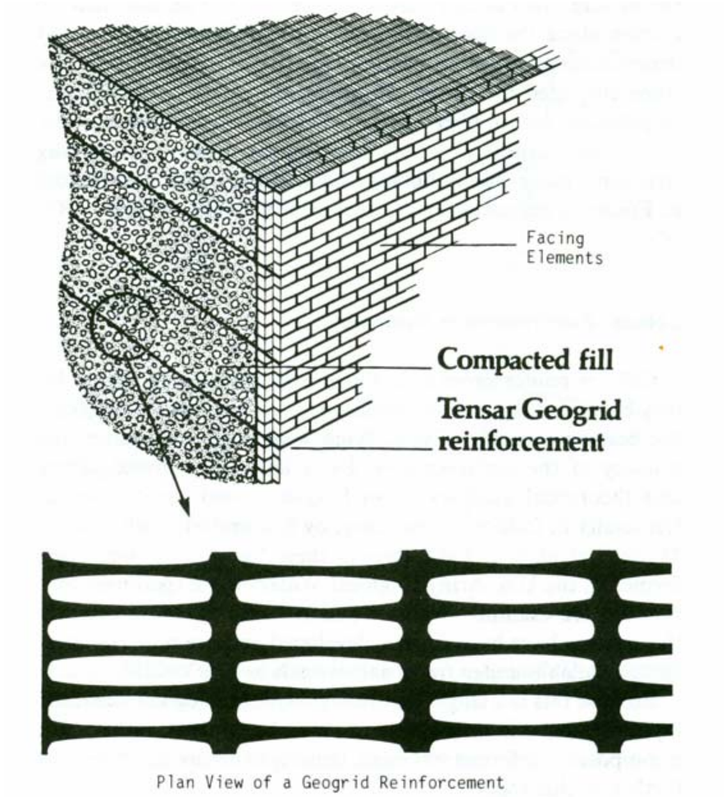


Figure 10.4 Schematic Example of Geogrid Reinforced MSE Wall (NCHRP, 1987)

ADVANTAGES (FHWA, 1998)

The principal advantage of MSE systems is cost, For walls > 5 m, cost savings of 20 to 60% over traditional walls have been achieved. The total cost of the MSE structure is dependent upon the costs of the facing, reinforcement, and soil backfill.

Compared with conventional concrete gravity and cantilever walls, other MSE advantages are:

a. Simple and rapid construction procedures that do not require large construction equipment of skilled labor

b. Less site preparation

c. Less space needed in front of the structure during construction operations

d. Reduced right-of-way acquisition

e. Because they are not rigid, they are tolerant to deformations, and therefore do not require strong foundation support

f. They are technical feasible to heights in excess of 25 m

g. Earthquake resistant

h. Precast concrete facing panels can have architectural surfaces to create an aesthetic appearance

i. MSE maximum differential settlements = 1/100 whereas conventional walls limited to 1/300 to 1/500. Full height panels limited to 1/500. Joints can be left between panels to accommodate differential settlements 13mm joint = 1/200.

The general disadvantages with MSE systems are:

a. A relatively large space behind the wall facing may be required to provide sufficient reinforcement length for design shear transfer, and accommodation of the assumed design failure surface.

b. MSE systems require select granular backfill. At sites lacking sufficient granular soils the cost of suitable backfill may render the system uneconomical.

c. Suitable design criteria are required to mitigate the corrosion of steel reinforcement ; these include galvanizing, and selection of non-corrosive backfill.

d. Polymer reinforcement may require suitable design criteria to prevent degradation in the ground, and creep effects.

e. Many MSE wall systems have proprietary features Hence, the successful construction requires a shared responsibility between material suppliers, design engineers, and contractors in a domain often dominated by structural engineers.

CONSTRUCTION SEQUENCE (PASSE, 2000, AND FHWA, 1998)

The following is a general sequence for the construction of MSE walls. Specific projects will vary from this sequence as required.

a. Subgrade preparation and placement of leveling pad. Subgrade preparation consists of removing unsuitable materials (organic soils, vegetation, etc), and compaction. An unreinforced leveling pad about 300 mm (12-inches) wide and 150mm (6-inches) thick is placed on the subgrade.

Prepare subgrade, compact , and excavate footing

Place concrete leveling pad

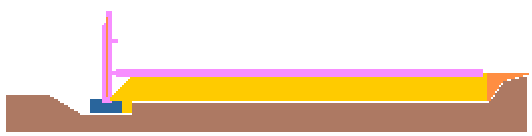
b. Erection of first row of panel. The panels may be full , and or half height. External bracing most likely will be used for stability. A batter of 1/8” per ft. is typically used for the 2nd and subsequent rows. Filter fabric is now adhered to the exposed panel using an adhesive.

Install, brace and place panels

c. Placement and compaction of 1st soil lift to 1st level of reinforcement. – Fill is usually compacted to 95% AASHTO T-99. The reinforced backfill should be dumped parallel to the wall and about the middle and then bladed forward toward the front face. The compacted fill should be slightly higher than the panel.

Compact 6” lifts to panel height

d. Placement of 1st layer of reinforcement on the backfill. The reinforcement is placed and connected perpendicularly to the facing panels.

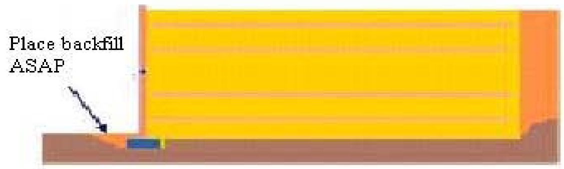


Place, connect and tighten reinforcement. Backfill against 1st panel after placement of reinforcement. Subsequent lifts can be placed directly against panel.

e. Placement of backfill over reinforcement. Backfill is placed over the reinforcement in 6” lifts. The backfill is placed approximately 3-ft from and parallel to the front panel and then bladed perpendicularly to the wall over the reinforcement. Once this is complete, the interior 3-ft of backfill are placed. Compaction equipment rolls parallel to the wall, first about 3-ft from the wall to the end of the reinforcement, and then the 3-ft next to the wall.

Continuation of fill placement

f. Construction of Coping. Coping is placed as the final construction sequence, and backfill placed in front shortly after completion of the 1st panel.



REINFORCED BACKFILL MATERIALS

MSE wall require high quality backfill for durability, good drainage, compactability, and high shear transfer from the reinforcement. Consequently, cohesionless materials are used and soils with high clay content are eliminated. FHWA (1998) recommends the following gradation limits for backfill in the reinforcement zone for steel reinforcement (Table 10.2). However for extensible reinforcement, FHWA (Table 10.3) recommends a maximum particle size of only 20mm (3/4-inch).

Table 10.2 Select Backfill-Inextensible Reinforcement

US Sieve size	Percent Passing
4inch (102mm)	100
No. 40 (.425mm)	0-60
No 200 (.075mm)	0-15

The PI shall be less than 6 (PI < 6).

Table 10.3 Select Backfill – Extensible Reinforcement

(Tensar Specs, www.tensarcorp.com, 2004)

US Sieve Size	Percent Passing
2 – inch (50mm)	100 - 75
¾ - inch (19mm)	100 – 75
No 4 (4.76 mm)	100 - 20
No 40 (0.425mm)	0- 60
No 200 (0.075mm)	0 -35

FDOT (2004) makes no distinction for backfill for extensible or inextensible reinforcement, as listed in Table 10.4

Table 10.4 Select Backfill for MSE Walls – FDOT (2004)

US Sieve	Percent Passing
3 1/2 inches [90 mm]	100
3/4 inch [19.0 mm]	70-100
No. 4 [4.75 mm]	30-100
No. 40 [425 µm]	15-100
No. 100 [150 µm]	5-65
No. 200 [75 µm]	0-15

PI < 6 and LL < 15

The design of buried steel reinforcement is predicated on backfills exhibiting minimum detrimental electrochemical properties; consequently, Table 10.5a lists FHWA and FDOT recommended electrochemical properties when using steel reinforcement, while Table 10.5b lists electrochemical properties for extensible reinforcement.

Table 10.5a Electrochemical Properties - Steel Reinforcement

Property	Criteria
Resistivity	> 3000 ohm-cm
pH	>5 <10
Chlorides	<100 ppm
Sulfates	<200 ppm
Organic content	1% max

FYI: Coke-cola pH = 4

Table 10.5b Electrochemical Backfills – Geosynthetics

Base Polymer	Property	Criteria
Polyester (PET)	pH	> 3 <9
Polyolefin (HDPE)	pH	> 3

THEORY (FHWA, 1998, NCHRP, 1987, AND Hausmann, 1990)

A reinforced soil mass is similar to a reinforced concrete structure, in that both concrete and soils are weak in tension, and thus the mechanical properties of the mass are improved by providing reinforcement. However, for soils the stress transfer between soil and reinforcement is frictional, whereas for concrete the stress transfer is chemical through the cement/reinforcement bond strength. The soil stress transfer mechanism is due to: (1) frictional, and (2) passive resistance. Friction is paramount for ribbed steel strips and longitudinal bars, whereas passive resistance is considered the primary interaction for geogrids, and wire mesh reinforcements.

The strength of reinforced soil can be interpreted in terms of Mohr-Couolmb theory, whereby the reinforcement increases the cohesion component (Figure 10.5, Schlosser, and Long, 1972). Considering that the unforced circle represents the *Active* earth pressure, then C_R can be shown to be a function of the reinforcement breakage stress, σ_R, as:

$$C_R = \frac{\sigma_R}{2\sqrt{K_a}}, \text{ where } K_a = \tan^2(45+\phi/2)$$

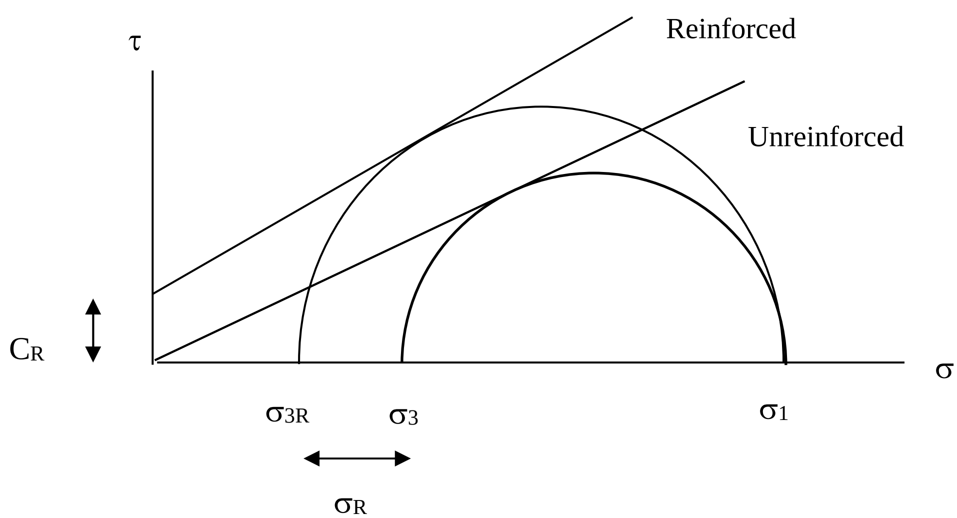


Figure 10.5a Mohr-Coulomb Representation of Reinforcement "Cohesion"

In the case where pullout (friction) is the controlling factor between the soil and reinforcement, then σ_R is proportional to the normal stress, σ_{1i}, With $\sigma_R = F\,\sigma_{1i}$, an increased friction angle develops, as; (Hausmann, 1990) $\sin\phi = \frac{1+F-K_A}{1-F+K_A}$, F is a friction factor. When $F = K_A$, the limiting value of $\phi_R = 90°$ occurs, meaning that failure is by breakage and not pullout (friction). Figure 10.5b illustrates this concept, where the Mohr-Coulomb failure line is bi-linear, with the initial portion controlled by friction, and the higher stress controlling breakage.

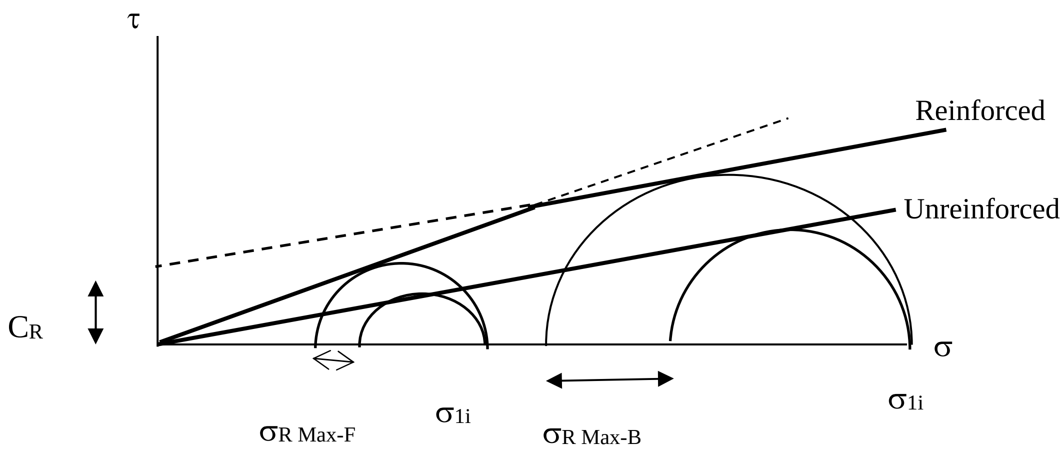


Figure 10.5b Mohr-Coulomb Representation of Friction Component of Reinforcement

DESIGN PROCEDURE FOR MSE WALLS

The general design considerations for MSW walls is divided into 2 areas:

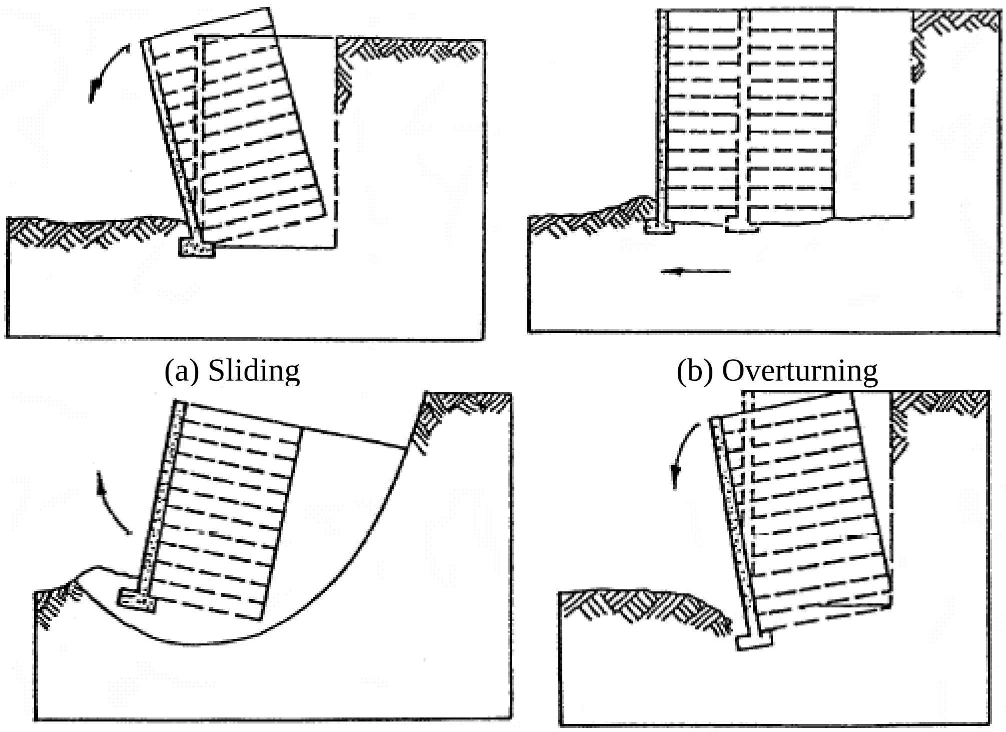
1. *External Stability* – External *stability* checks involve; (a) sliding, (b) overturning, (c) bearing capacity, and (d) overall deep seated stability. (See Figure 10.6)
2. *Internal Stability* – Internal stability checks involve; (a) pullout of reinforcement due to friction failure, and (b) rupture or breakage of the reinforcement due to tension failure.

PRELIMINARY SIZING (FHWA, 2001)

A preliminary reinforcement length should be chosen that is greater than 0.7H and 2.5 m, where H is the design height of the structure. Structures with sloping surcharge fills or other concentrated loads, as in abutment fills, generally require longer reinforcements for stability, often on the order of 0.8H to as much as 1.1H.

Minimum embedment depth at the front of the wall is as follows:

Slope in Front of Wall	Minimum D (top of leveling pad)
Horizontal (walls)	H / 20
Horizontal (abutments)	H / 10
3H : 1V	H / 10
2H : 1V	H / 7
3H : 1V	H / 5



(a) Sliding (b) Overturning

(c) Bearing Capacity (d) Deep Seated Failure

Figure 10.6 External Stability Considerations for MSE Walls (from FHWA, 2001)

Horizontal and vertical spacing of the reinforcement is dependent upon the location of the wall panel connections supplied by the manufacturer. Reinforced Earth® cruciform shaped panels have horizontal and vertical spacings at 2.5-ft (0.75m).

LATERAL EARTH PRESSURES (FHWA, 1990)

Coulomb earth pressure theory is used for MSE wall design calculations. However, based upon finite element studies the lateral thrust has been found to incline downward at an inclination angle λ, instead of β (no wall friction) or δ (wall friction), with λ defined as

$\lambda = \left[1.2 - \frac{L}{H}\right] \phi_b$ when reinforcements are inextensible

$\lambda = 0$ when reinforcements are extensible

Active earth force, P_a is $P_a = \left(\frac{\gamma H'^2}{2} + qH'\right)K_a$, where K_a is given by;

$$K_a = \left[\frac{\sin(\theta - \phi_b)\ /\sin\theta}{\sqrt{\sin(\phi + \lambda)} + \sqrt{\sin(\phi_b + \lambda)\ \sin(\phi_b - \beta)\ /\ \sin(\theta - \beta)}}\right]^2$$

External Stability Calculations

Assuming the reinforced soil mass can be represented as a block; then:

a. **Sliding:** $FS_{sliding} = \dfrac{\Sigma \text{ Sliding Resistance Forces}}{\Sigma \text{ Sliding Driving Forces}}$ $\geq$ 1.5 w/o passive pressure at toe

2.0 w/ passive pressure at toe

b. **Overturning:** $FS_{overturrning} = \dfrac{\Sigma \text{ Resistance Moments}}{\Sigma \text{ Overturning Moments}} \geq 2$

c. **Location of Normal Resultant on Base:** Must be within middle 1/3 of base

d. **Bearing Capacity of Base:** $FS_{BC} = \dfrac{\text{Ultimate Bearing Pressure}}{\text{Max. Base Bearing Pressure}} \geq 2.0 \text{ or } 2.5$

e. **Deep-Seated Shear Failure:** slope stability analysis, more important for cohesive soils and/or when weak strata found under wall (earthquake forces and liquefaction also)

f. **Settlement:** especially check consolidation of cohesive soils (some walls are supported on piles to avoid bearing capacity and settlement problems)

g. **Horizontal Displacement:** determine tolerable horizontal displacement based upon face batter $\delta_h \leq$ ¾" per 10 ft. height (6.2 mm/m) for precast panels. (See Figure 10.7)

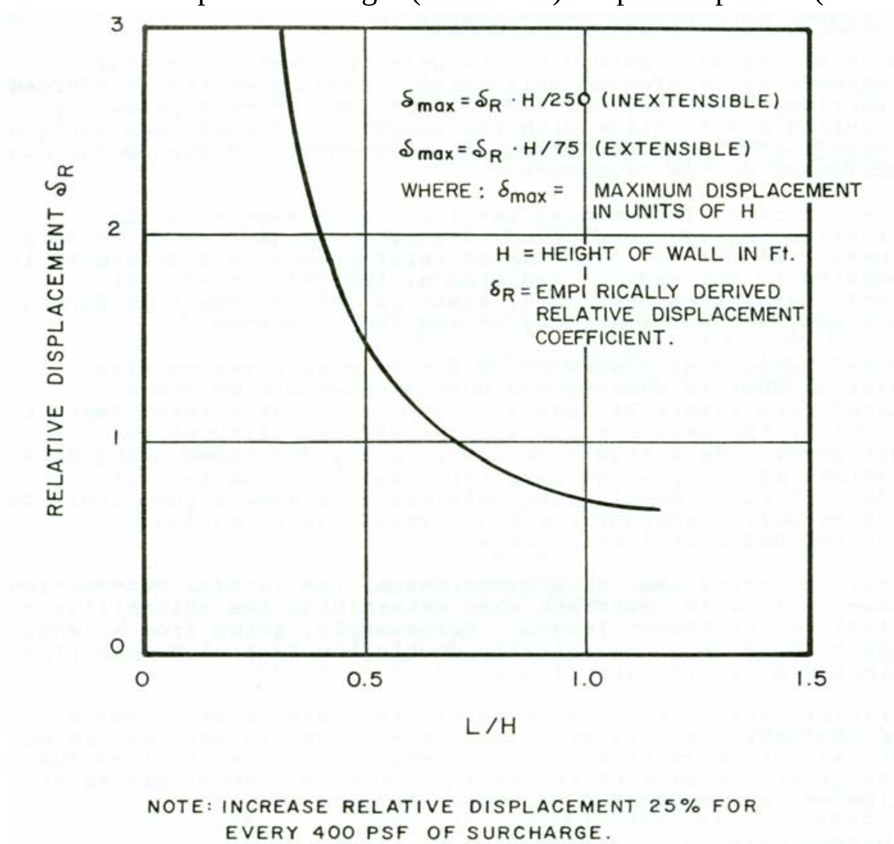
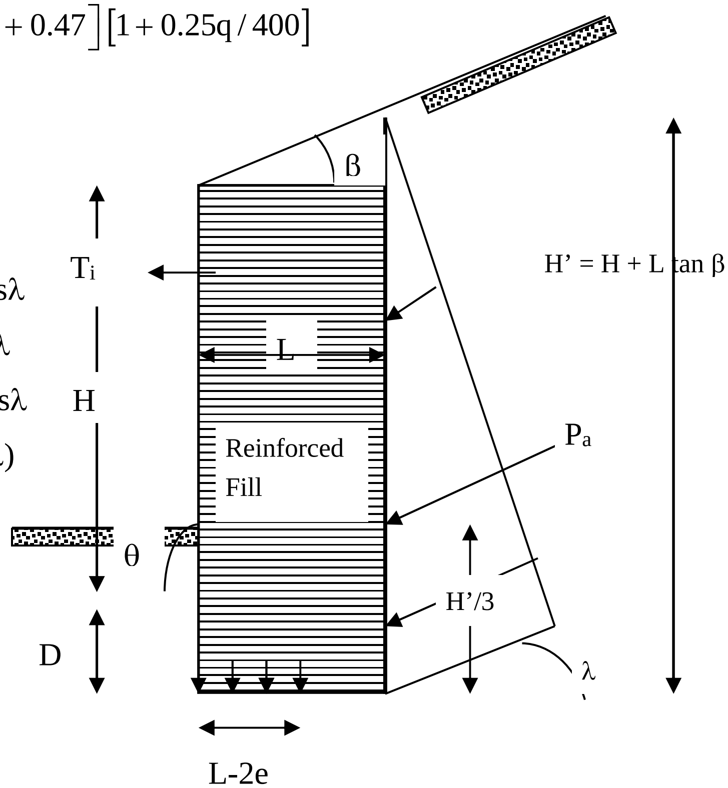


Figure 10.7 Empirical Curve for Estimating Settlement (from FHWA, 1990)

$$\delta_R = \left[0.282(L/H)^{-1.838} + 0.47\right]\left[1 + 0.25q/400\right]$$

Example 1 Block



1. Vertical Components $P_{av} = P_a \text{Sin}\ \lambda$
2. Horizontal Components $P_{ah} = P_a \text{Cos}\lambda$
3. Vertical Components $P_{pv} = P_p \text{Sin}\ \delta\lambda$
4. Horizontal Components $P_{ph} = P_p \text{Cos}\lambda$
5. $R = \Sigma\ V = W + P_a \sin(\lambda) - P_p \sin(\lambda)$

   $T = C_a + R \tan \delta_B = c_a B + R \tan \delta_B$

   (c_a is adhesion < cohesion

   δ_B is base-soil friction angle)

- **Overturning:** sum moments about toe (R = 0)

$$FS_{overturrning} = \frac{\Sigma \text{ Resistance Moments}}{\Sigma \text{ Overturning Moments}} = \frac{\Sigma M_r}{\Sigma M_o}$$

$$= \frac{W(L/2) + P_a \sin(\lambda)(L) + P_{ph}(D/3)}{P_a \cos(\lambda)(H'/3)}$$

- **Sliding:**

$$FS_{sliding} = \frac{\Sigma \text{ Sliding Resistance Forces}}{\Sigma \text{ Sliding Driving Forces}}$$

$$= \frac{P_p \cos(\lambda) + T}{P_a \cos(\lambda)}$$

- **Bearing:** sum moments about A to find distance x_R to resultant R and then check q_{max} using flexural analysis ($\sigma \pm Mc/I$).

$$x_R = \frac{\Sigma M_r - \Sigma M_o}{R}$$

$$\text{eccentricity } e = \left(\frac{L}{2} - x_R\right) \leq \left(\frac{L}{6}\right) \text{to be in center 1/3 of base}$$

$$q = \left(\frac{R}{L}\right) \pm \left(\frac{Re(L/2)}{L^3/12}\right) = \left(\frac{R}{L}\right) \pm \left(\frac{6\,Re}{L^2}\right)$$

Internal Stability

Horizontal Tensile Force: The horizontal tensile force, T_i, for each layer is given by:

$T_i = K\sigma_v' S_h S_v$ where $\sigma_v = (\gamma Z_i + q)$, and K is presented in Figure 10.8. Note that K_R is the Rankine Earth Pressure Coefficient, and not K_a (Coulomb) as used for calculating the Active force.

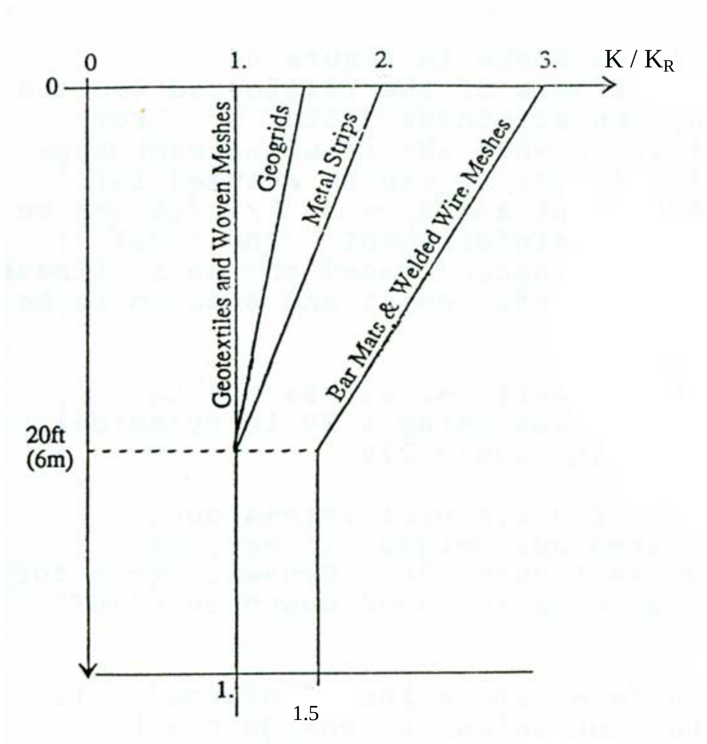


Figure 10.8 Variation of the Stress Ratio K with Depth

Internal Stability with Respect to Breakage

Stability with respect to breakage requires that:

$$T_{allowable} = \frac{F_y A_c}{b} FS$$

where FS = 0.55 for steel reinforcement strips

b = gross width of strip, or geotextile sheet

A_c = anticipated cross-section area after corrosion losses for design life

Corrosion rates:

For zinc galvanization 15μm / yr. (first 2 years), 4 μm / year (thereafter)
For carbon steel 12μm / year (thereafter)

Selection for T_a for polymer geogrids is more difficult than for steel, as the tensile properties are affected by creep, installation damage, aging, temperature, and confining stress. Consequently, a conservative reduction factor (FS) of 1/7 is suggested.

Corrosion example: Consider a 4 mm (4000 μm) thick x 50 mm (1.969") wide steel strip with 3.4 mils galvanization (86 μm/side), find $T_{allowable}$ for a design life = 100 years

$$T_{allowable} = \frac{F_y A_c}{b} FS$$

A_C Service life of zinc: 1st 2 years (15 μ/yr) = 30 μ
Thereafter: (86-30) μm = 56 μm/4 μm/yr = 14 years
Total zinc life = 2 + 14 = 16 years

Thickness of reinforcement after 100 years

T_{100} = 4000μm – (2 sides)[12 μm] x (100-16 yrs) = 1984 μm
A_{c100} = 50 mm x 1.984 mm = 99.0 mm^2 = 0.1535 in^2

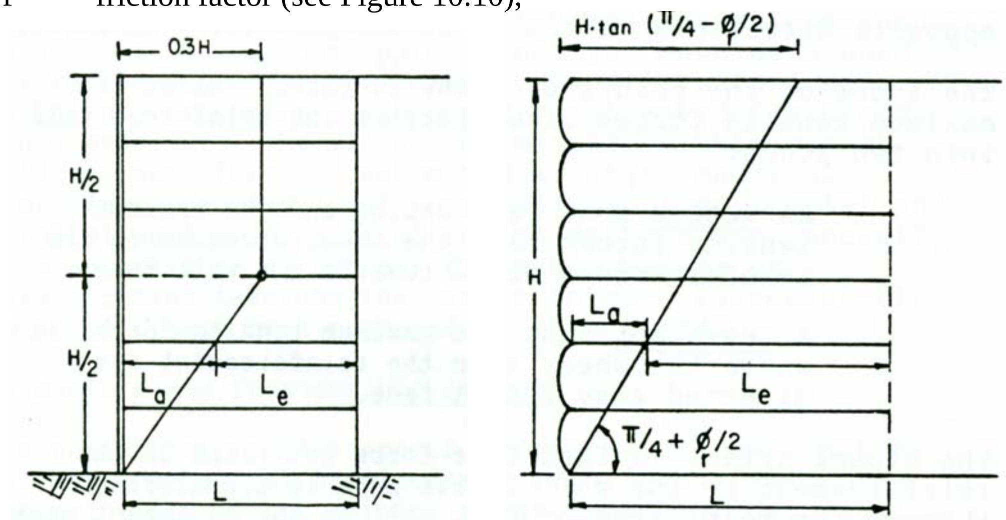
$$T_{allowable} = \frac{F_y A_c}{b} FS = \frac{(65ksi)(0.1535in^2)}{1.969"}(0.55) = 2.787 \text{ kips / ln in.}$$

Internal Stability with Respect to Pullout (Friction)

Internal stability with respect to pullout of the reinforcements requires that sufficient length, L_e , be available beyond the potential failure plane to ensure stability; thus:

$$T_{allowable} = \frac{2L_e \, w \, \sigma_v \, f^*}{FS}$$

where L_e = effective length beyond the failure surface (see Figure 10.9)
2 = both sides of reinforcement
w = strip width
σ_v = vertical stress at level of strip
f^* = friction factor (see Figure 10.10),



(a) Inextensible (b) Extensible

Figure 10.9 Failure Surface for MSE Reinforcements

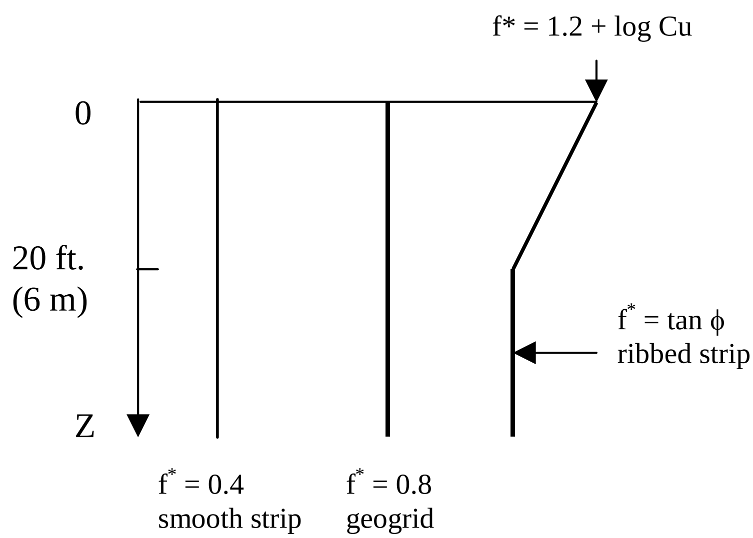
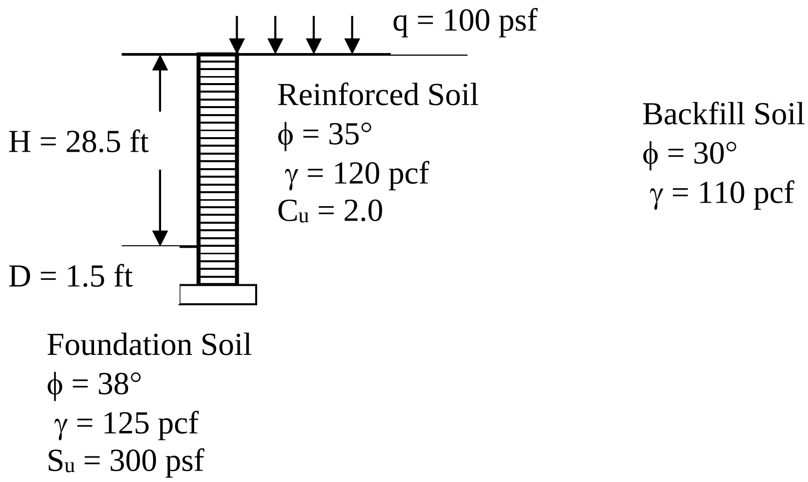


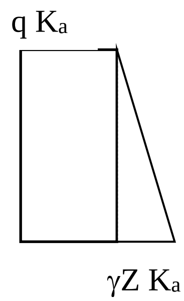
Figure 10.10 Friction (f^*) Values (C_u = uniformity coefficient, D_{60}/D_{10})

MSE WALL DESIGN EXAMPLE

Determine the external and internal stability FS's for a 30 ft high MSE wall, supporting a traffic surcharge of 100 psf. Original estimates are: strip length = 18 ft. and $S_h = S_v$ = 2.5 ft., and design life = 100 years.



EXTERNAL STABILITY



1. Calculate Active earth force, P_a

$$P_a = \left[q H + \frac{\gamma_b H^2}{2} \right] K_a$$

a. $K_a = \left[\dfrac{\mathrm{Sin}(\theta - \phi_b)/\sin(\theta)}{\sqrt{\sin(\theta + \lambda)} + \sqrt{\sin(\phi_b + \lambda)\sin(\phi_b - \beta)/\sin(\theta - \beta)}} \right]^2$

b. $\lambda = \phi_b \left[1 - (1 - \beta/\phi_b)(L/H - 0.2\right] = 30\left[1-(1-0)(18/30-0.2)\right] = 18^\circ$

c. $K_a = \left[\dfrac{\mathrm{Sin}(90-30)/\sin(90)}{\sqrt{\sin(90+18)} + \sqrt{\sin(30+18)\sin(30-0)/\sin(90-0)}} \right]^2 = 0.2986$

d. $P_a = \left[100 \times 30 + \dfrac{110 \times 30^2}{2} \right] 0.2985 = 15{,}678 \text{ lbs}$

e. $P_{aH} = P_a \cos\lambda = 15{,}678\cos 18^\circ = 14{,}910 \text{ lbs} \;\&\; P_{aV} = P_a \sin\lambda = 15{,}678 \sin 18^\circ = 4{,}845 \text{ lbs}$

2. Calculate weight of reinforced soil and surcharge

a. $W_R = (120\,\text{pcf})\,(30\,\text{ft})\,(18\,\text{ft}) = 64{,}800 \text{ lbs} \;\&\; Q = 100\,\text{psf}\;(18\,\text{ft}) = 1800 \text{ lbs}$

3. Check the FS against sliding:

a. $FS_{sliding} = \Sigma Forces_{horizontal\ resisting} / \Sigma Forces_{horizontal\ driving} = \dfrac{(W' + P_{aV})\tan\phi_{R\,or\,fdn}}{P_{aH}}$,

$W' = W_R + Q = 64{,}800 + 1{,}800 = 66{,}600$ lbs

The possibility exists that sliding can occur in the reinforced soil or the foundation soil, whichever is smaller; hence:

Resisting Forces:

F_H = (66,600 + 4,845) tan 35 = 50,026 lbs or

$F_H = (W' + P_{aV}) \tan\phi_{fdn} + CL$ = (66,600 + 4845) tan 38 + 300 psf x 18ft = 61,219 lbs

$$FS_{sliding} = \frac{(50{,}026)\tan 35^\circ}{14{,}910} = 3.36$$

4. Check the FS against overturning:

a. $FS_{overturning} = \sum$ moments resisting / $\sum$ moments overturning

b. $\sum M_{resist} = W''\bar{X} + P_{aV}L = (66,600)(18/2) + 4845(18) = 686,610 \text{ ft} - \text{lbs}$

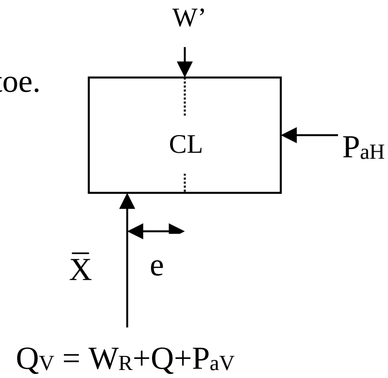
c. $\sum M_{overturn} = P_{aH} \times \bar{y} = \left[\dfrac{\gamma_b H^2}{2} K_a \times \dfrac{H}{3} + qHK_a \times \dfrac{H}{2}\right]\cos\lambda$

$$\sum M_{overturn} = P_{aH} \times \bar{y} = \left[\frac{110(30)^2}{2}(.2986) \times \frac{30}{3} + 100(30)(.2986) \times \frac{30}{2}\right]\cos 18^\circ = 153,301 \text{ ft} - \text{lbs}$$

$$\therefore, FS_{overturning} = \frac{686610}{153301} = 4.48$$

5. Calculate Eccentricity and Bearing Capacity:

a. Eccentricity calculated by taking moments @ toe.



or

$$\sum M_o - \sum M_R + Q_V\bar{X} = 0$$

$$\therefore \bar{X} = \frac{\sum M_R - \sum M_O}{Q_V} = \left(\frac{L}{2} - e\right)$$

$$\bar{X} = \frac{686610 - 153301}{71445} = 7.465$$

$$e = \frac{18}{2} - 7.465 = 1.54 \le \frac{L}{6}$$

b. Bearing capacity uses Meyerhof’s reduced footing size (L’ = L-2e) and neglects corrections for embedment and inclination.

1. L’ = L – 2e = 18 – 2 (1.54) = 14.93 ft
2. $q_V = Q_V / L' = 71,445 / 14.93 = 4,786$ psf
3. $q_u = CN_c + \dfrac{\gamma L'}{2} N_\gamma$,

   where

$$N_q = \exp\left[\pi \tan\phi_{fdn}\right](\tan^2(45 + \phi/2) = \exp\left[\pi \tan 38\right]\tan^2(64) = 48.93$$

$$N_c = (N_q - 1)\cot\phi_f = (48.93 - 1)\cot 38^\circ = 61.3$$

$$N_\gamma = (N_q - 1)\tan(1.4\phi) = (48.93 - 1)(\tan 53.2^\circ) = 64.1$$

Note: Meyerhof uses 1.4ϕ for plane strain conditions

$$q_u = 300\text{psf}(61.3) + \frac{125\text{pcf}(14.93\text{ft})(64.1)}{2} = 78,110 \text{ psf}$$

$$\therefore FS_{Bearing} = \frac{q_u}{q_V} = \frac{78,110}{4786} = 16.32$$

6. Check horizontal displacement (see Figure 10.7)

   a. $\delta = \delta_R \times \frac{H}{250}$, where $\delta_R = \left[0.282(L/H)^{-1.838} + 0.47\left[1 + 0.25q/400\right]\right]$

   $$\delta_R = \left[0.282(18/30)^{-1.838} + 0.47\left[1 + 0.25(100psf)/400\right]\right] = 1.26$$

   $$\delta = 1.26 \times \frac{30}{250} \times 12 = 1.82" \text{ and}$$

   $$\delta_{allowable} = (3/4")/10ft = 0.75 \times 30/10 = 2.25" \geq 1.82 \text{ OK}$$

***Internal Stability*:** Determine the internal stability for strips at: 13.75 ft and 28.75 ft. from wall top.

7. Find T_i at Z = 13.75 and 28.75 ft.

   a. Ti = (γZ + q) K SHSV

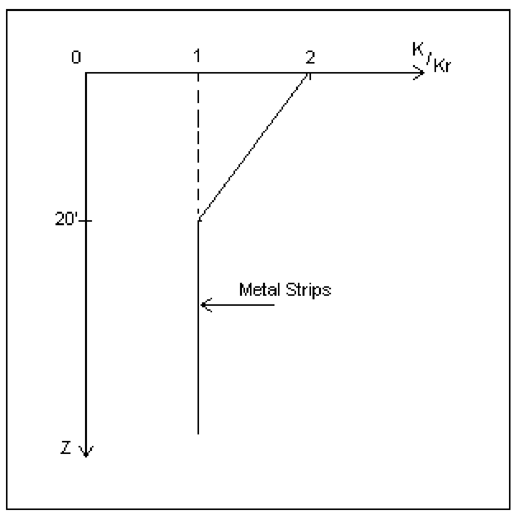
   b. At Z = 13.75 ft , K / KR (by interpolation) = 1+(20-13.75)/20 = 1.3125

   c. At Z = 28.75, K / KR = 1.0

   d. Note that KR is Rankine earth pressure or KR = tan2 (45-ϕ/2) =0.271

   e. At Z = 13.75ft., Ti = (120 pcf x 13.75ft + 100psf) (1.3125 x 0.271) (2.5 x 2.5ft) = 3,890 lbs.

   f. At Z = 28.75 ft., Ti = (120 pcf x 28.75ft + 100psf) (1.0 x 0.271) (2.5 x 2.5ft) = 6,013 lbs.



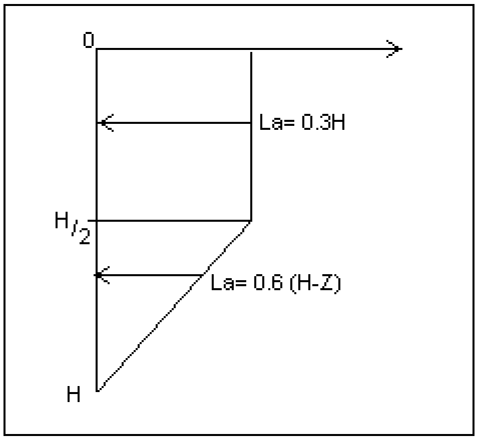
8. From previous example on corrosion for 100 year design life, A_c after corrosion = 0.1535 in^2, and $T_{allowable} = F_y A_c = (0.55 \times 65,000 \text{ psi})(0.1535 \text{ in}^2) = 5,487.6 \text{ lbs}$

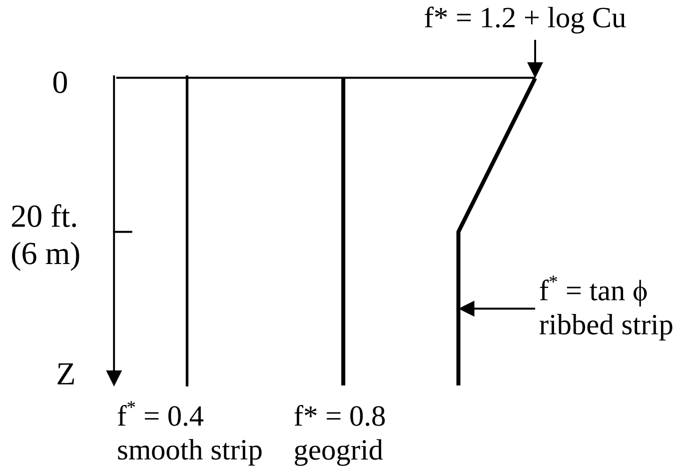
   a. $FS_{breakage}$ at 13.75 ft. = $T_{allowable}$ / T_i = 5,487.6 / 3,890 = 1.41

   b. $FS_{breakage}$ at 28.75 ft. = $T_{allowable}$ / T_i = 5,487.6 / 6,013 = 0.91

9. The $FS_{pullout}$ can be calculated as:

$$FS_{pullout} = \frac{2L_e w \sigma_v f^*}{T_i} \qquad \text{where:}$$





a. At Z = 13.75 ft., L_a = 0.3 (30ft) = 9.00 ft., and L_e = 18 – 9.0 = 9.0 ft.

   At Z = 28.75 ft., L_a = 0.6 (30 –28.75) = 0.75 ft. and L_e = 18 – 0.75 = 17.25

b. At Z = 13.75 ft. σ = (120 pcf x 13.75ft + 100psf) = 1750 psf

   At Z = 28.75 ft. σ = (120 pcf x 28.75ft + 100psf) = 3550 psf

c. The width, w is not consider in corrosion, only thickness, hence; 50 mm = 1.968”

d. The friction, f*, for ribbed strips for Z = 13.75 ft. 1.2 + log 2 = 1.5, and tan35 = 0.7, thus by interpolation f* = 0.95. For Z = 28.75ft., f* = tan 35° = 0.7

e. At Z = 13.75 ft $FS_{pullout} = \frac{2(9ft)(1.968/12)(1750psf)(0.95)}{3890} = 1.26$

f. At Z = 28.75 ft., $FS_{pullout} = \frac{2(17.25ft)(1.968/12)(3550psf)(0.70)}{6013} = 2.34$

Spreadsheet REWALL99

The following figures present a spread-sheet verification of the example problem.

REINFORCED EARTH FILL WALL DESIGN SPREADSHEET

Eloy D. Briceño
UF, Julio 1994
Modified by Roberto Fernandez, March ' 99

FOR SUMMARY TABLE ---------->>>>

SOIL PROPERTIES

	Unit weight pcf	Friction angle deg	Cohesion psf
Reinforced soil	120.00	35.00	
Backfill soil	110.00	30.00	
Foundation soil	125.00	38.00	300.00

WALL DIMENSIONS

Height, H ft	Slope angle deg	Face batter deg	Slope at toe deg	Vert. surcharge psf
30.00	0.00	90.00	0.00	100.00

Bar length L, Pies 18.00

A.- EXTERNAL STABILITY

A.1.- Factor of Safety against sliding
A.2.- Factor of Safety against overturning
A.3.- Excentricity
A.4.- Factor of Safety on bearing capacity

	Lambda deg	Ka	Pa lbs	Pav lbs	Pah lbs	W' lbs	Qv lbs
	18.00	0.2986	15,677.5	4,844.6	14,910.2	66,600.0	71,444.6
FHWA =	18.00						

Fh lbs	Nc	Nq	N_γ	qv psf	qu psf	X' ft
50,026.1	61.35	48.93	64.07	4786.14	78,183.9	7.46

Mr	Mo
686,603.3	153,362.3

Exc. ft	FS Sliding	FS Overturning	FS B. Capac.	Horiz. Displac. delta (in)
1.54	3.36	4.48	16.34	1.82

Eloy D. Briceño
UF, Julio 1994

SUMMARY TABLE

A.- EXTERNAL STABILITY

A.1.- Factor of Safety against sliding
A.2.- Factor of Safety against overturning
A.3.- Excentricity
A.4.- Factor of Safety on bearing capacity
A.5.- Maximum Horizontal Displacement

FS Sliding	FS Overturning	Eccentricity ft	FS Bearing Capac.	Horiz. Displac. delta (in)
3.36	4.48	1.54	16.34	1.82

RECOMMENDED VALUES

FS Sliding	FS Overturning	Eccentricity ft	FS Bearing Capac.	Max. Horiz. Displac. (in)
1.50	2.00	3.00	2.00	2.25
OK !!!	OK !!!	OK !!!	OK !!!	OK !!!

SUMMARY TABLE

B.- INTERNAL STABILITY

B.1.- Factor of Safety against breakage
B.2.- Factor of Safety against pullout

FS Breakage	Pullout Factor of Safety
0.91	1.26

Breakage Factor of Safety	Pullout Factor of Safety
1 *	1.50
WARNING !!!	WARNING !!!

* THESE VALUES HAVE A FACTOR OF SAFETY OF 1.82 (1/0.55)

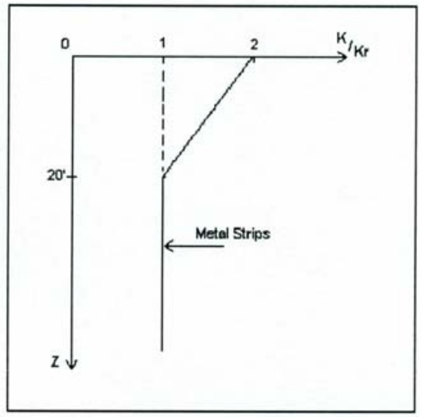
B.- INTERNAL STABILITY

B.1.- Bar corrosion

B.2.- Factor of Safety against breakage

B.3.- Factor of Safety against pullout

Design life, yrs		100.00
Thickness of galvanization, μm/side	(86.00)	86.00
Thickness of steel bar, mm		4.00
Width of steel bar, mm		50.00
Rate of corrosion:		
-Steel μm/yr	(12.00)	12.00
-Zinc: first two years, μm/yr	(15.00)	15.00
Thereafter, μm/yr	(4.00)	4.00
Service life of zinc, years		16.00
Thickness of reinforcement at service life, mm		1.98
Steel Fracture stress, psi		65,000
Steel yield stress, psi		35,750
Bolt hole diameter, in		0
Bar area at connection, Ac, in²		0.154
Vertical spacing, Sv, ft		2.50
Horizontal spacing, Sh, ft		2.50
Tyield/strip, lbs		5,497
Tyield/unit length of wall, plf		2,199

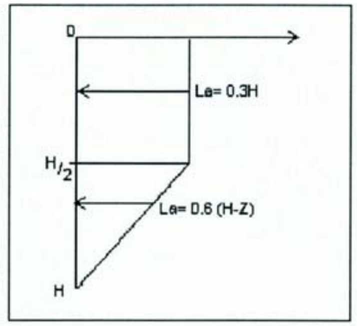


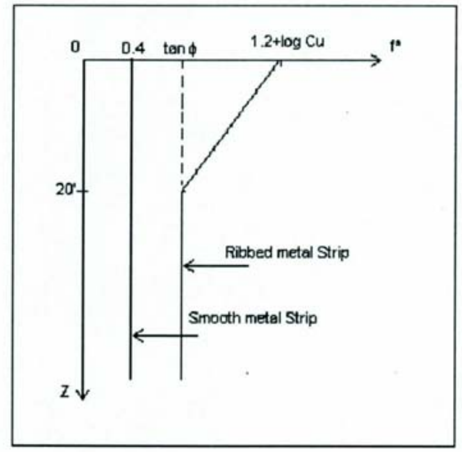
Active lateral pressure coefficient, Kr	0.271

Layer N°	Depth, Z ft	K/Kr	K	σ_v psf	σ_h psf	Tmax plf	Break. Connec. Factor of Safety
1	1.25	1.938	0.525	250.00	131.26	820.4	6.70
2	3.75	1.813	0.491	550.00	270.14	1688.4	3.26
3	6.25	1.688	0.457	850.00	388.70	2429.4	2.26
4	8.75	1.563	0.423	1150.00	486.94	3043.3	1.81
5	11.25	1.438	0.390	1450.00	564.84	3530.3	1.56
6	13.75	1.313	0.356	1750.00	622.43	3890.2	1.41
7	16.25	1.188	0.322	2050.00	659.69	4123.1	1.33
8	18.75	1.063	0.288	2350.00	676.63	4228.9	1.30
9	21.25	1.000	0.271	2650.00	718.12	4488.3	1.22
10	23.75	1.000	0.271	2950.00	799.42	4996.4	1.10
11	26.25	1.000	0.271	3250.00	880.72	5504.5	1.00
12	28.75	1.000	0.271	3550.00	962.01	6012.6	0.91
13							

Ribbed metal strip

Coef. Uniform.		2.00
Scale effect factor, alpha	(1)	1
Bar shape factor, C	(2)	2
Minimum friction factor, f*min. *		0.700
Maximum friction factor, f*max. *		1.501





Layer Nº	Depth, Z ft	Friction Factor f*	Active Length La, ft	Effective Length Le, ft	Pullout Resistance per strip lbs	Pullout Resistance per wall ft plf	Pullout Factor of Safety
1	1.25	1.451	9	9	1,071.1	428.44	1.31
2	3.75	1.351	9	9	2,193.8	877.54	1.30
3	6.25	1.251	9	9	3,139.2	1255.69	1.29
4	8.75	1.151	9	9	3,907.3	1562.91	1.28
5	11.25	1.051	9	9	4,498.0	1799.19	1.27
6	13.75	0.950	9	9	4,911.3	1964.52	1.26
7	16.25	0.850	8.25	9.75	5,576.2	2230.49	1.35
8	18.75	0.750	6.75	11.25	6,507.4	2602.96	1.54
9	21.25	0.700	5.25	12.75	7,761.7	3104.69	1.73
10	23.75	0.700	3.75	14.25	9,656.9	3862.77	1.93
11	26.25	0.700	2.25	15.75	11,758.9	4703.56	2.14
12	28.75	0.700	0.75	17.25	14,067.6	5627.04	2.34
13							

COST INFORMATION

MSE wall costs are mainly composed of: facing panels, reinforcement, and select backfill. It has beed found that MSE walls with precast concrete facings are usually less expensive that reinforced concrete retaining walls for heights greater than 10 – 15 feet. Segmental block walls are competitive with concrete walls at heights less than 15 ft.(FHWA 116)

For segmental precast MSE walls, typical costs are:

Erection of panels	20-30 percent
Reinforcement	20-30 percent
Facing panels	25-30 percent
Backfill materials	35-40 percent

CALTRANS costs (2002) were:

MSE walls	$21/ft^2$
Backfill	$25/yd^3$
Soil Nails	$55/ft^2$

REFERENCES

Florida Department of Transportation (2004), Section 548 Retaining Wall Systems, http://www.dot.state.fl.us/specificationsoffice/2004BK/toc.htm

FHWA (1998) "Ground Improvement Technical Seminars – Demo. Project 116," Publ.No. FHWA –SA-98-086, Authors: Elias, V., Welsh, J., Warren, J., and Lukas, R.

FHWA (1990) "Reinforced Soil Structures Vol. 1. Design and Construction Guidelines," Publ. No. FHWA-RD-89-043.

FHWA (2001), Mechanically Stabilized Earth Walls and Reinforced Soil Slopes Design and Construction Guidelines, FHWA Report No. FHWA-NHI-00-043, Authors: Elias Victor, Christopher, Barry, and Berg, Ryan.

Hausmann, M.R. (1990) Engineering Principles of Ground Modification, McGraw Hill Publishers.

NCHRP (1987) Reinforcement of Earth Slopes and Embankments, NCHRP report 290: Author: J.K. Mitchell and W.B. Willet

Passe, P.D. (2000), "MSE Walls Inspectors Handbook," FDOT, Tallahassee, Florida

Schlosser, F. and Long, N.C. (1972) "La Terre Armee dans L'Echaguer de Sete," Revue Generale des Rates et des Aerodromes, No. 480, reported in NCHRP (1987)

Tensar Specs, www.tensarcorp.com,2004.

Chapter 11

REINFORCED SOIL SLOPES

INTRODUCTION

Reinforced Soil Slopes (RSS) are a viable option to using high cost retaining walls or space hungry flat sloped embankments. As shown in Figure 11.1, layers of reinforcement can be placed in the slope during construction or reconstruction, which provides steeper slopes. Essentially, reinforced slopes evolved from except erosion protection and secondary reinforcement replace the MSE wall facing panels. however, the first reported use of reinforced steepened slopes is believed to be the west embankment for the great wall of China. The highest constructed RSS structure in the U.S. to date has been 43 m (141 ft). (Elias, 2001) Typically reinforced slopes are inclined less than 70° and geotextiles and geogrids are used as reinforcement.

In highway construction, roads may be widened over existing flatter slopes without encroaching upon existing right-of-way, or requiring retaining wall construction (Elias, 1998). In the case of slide repair, use of existing slide debris instead of importing a higher quality backfill may produce savings. Figure 11.2 presents the RSS used at Seabreeze Bridge, Daytona Beach, FL.

There are two main reasons for using RSS. (Elias, 1998):

1. To provide a steeper than *safe* unreinforced slope, or repair a failed slope.
2. To provide improved compaction along the slope face, thus decreasing the tendency for erosion and sloughing.

Additional improvements have been found in cohesive soils by using geotextiles to improve internal drainage that permit rapid pore pressure dissipation of the compacted soils.

ADVANTAGES

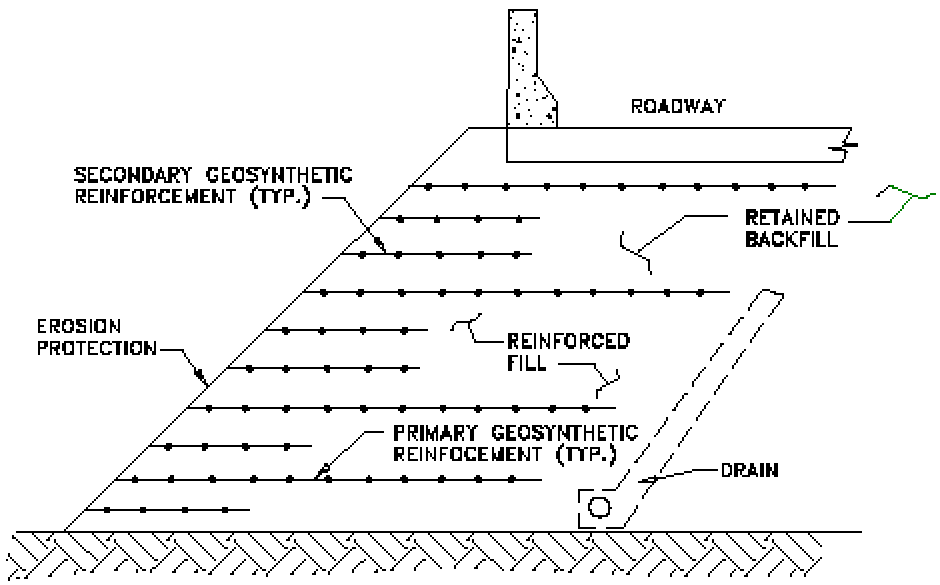
Economically, a safe steeper RSS than normally possible saves embankment material, and right-of-way costs.

Economically, repair of failed slopes reusing slide debris instead of higher quality backfill saves costs.

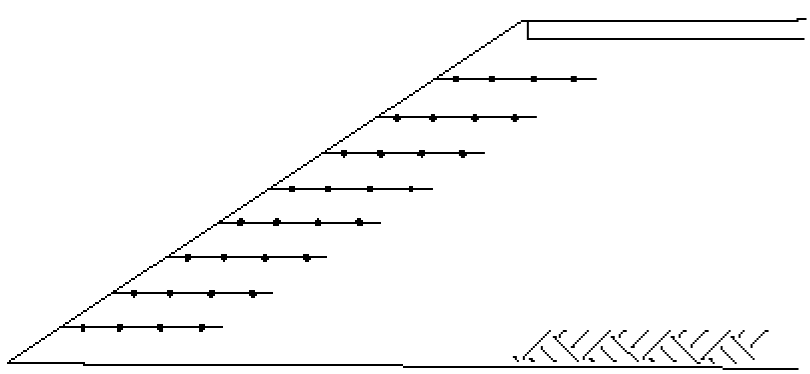
Aesthetically, use of vegetated slopes that blend with the natural environment can be more pleasing than retaining walls. Additionally, RSS is a better sound attenuator than concrete

A fairly wide range of backfill soils can be used unlike select backfill for MSE structures.

Polymer geogrids and geotextiles are chemically inert, and unaffected by deicing salts. Additionally, they are light-weight and easy to handle.



(a)



(b)

Figure 11.1 Examples of RSS (from Elias, 2001)

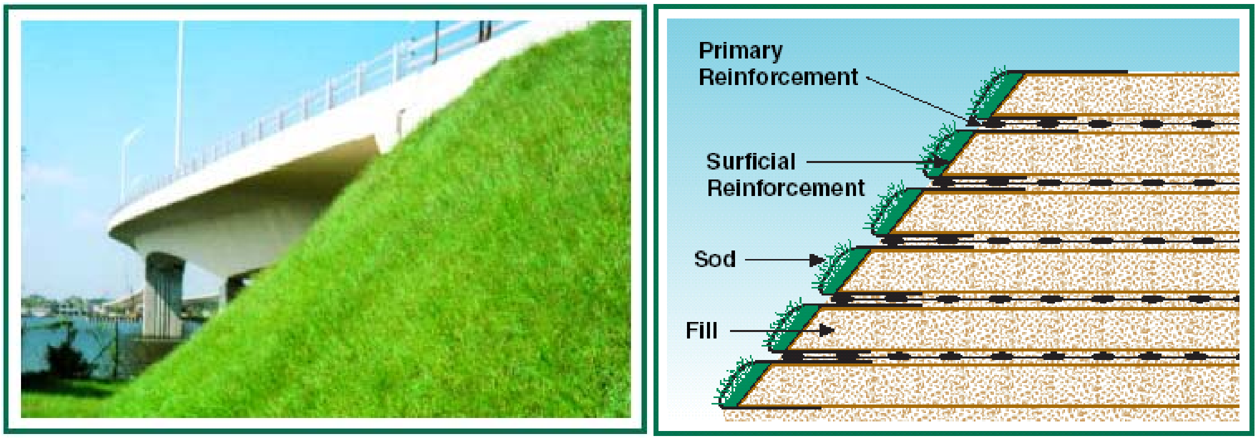


Figure 11.2 Reinforced Slopes – Sea Breeze Bridge Approach – Daytona Beach, FL

(from Tensar)

The deformation response and absence of concrete panels provide RSS with flexibility to absorb large deformations created by poor foundations or seismic action.

Construction is quicker as temporary bracing, forms, and alignment of concrete panels is avoided.

DISADVANTAGES

RSS slopes are constructed and limited to embankments, not cuts.

Steep vegetated slopes may cause maintenance (grass mowing) problems.

Near surface erosion/sloughing protection is critical to the performance of RSS. Intermediate reinforcement, drainage, and selection of proper vegetation type for growth must be considered.

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. (Elias, 2001)

Backfill Recommendations (Elias, 2001)

For RSS structures, less select backfill than MSE walls can be used as facings are typically flexible and can tolerate some distortion during construction. Even so, a high quality embankment low plasticity fill meeting the following gradation requirements to facilitate compaction and minimize reinforcement requirements is recommended. The following guidelines shown in Table 11.1 are provided as recommended backfill requirements for RSS construction:

Table 11.1 Recommended Backfill Requirements for RSS Construction (Elias, 2001)

Sieve Size	Percent Passing
20 mm*	100
4.76 mm (No. 4)	100 –20
0.425 mm (No. 40)	0 – 60
0.075 mm (No. 200)	0 – 50
Plasticity Index (PI) < 20	

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

GEOSYNTHETIC REINFORCEMENT (Elias, 2001)

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

Polymeric reinforcement, although not susceptible to corrosion, may degrade due to physicochemical activity in the soil such as hydrolysis, oxidation, and environmental stress cracking depending on polymer type. In addition, these materials are susceptible to installation damage and the effects of high temperature at the facing and connections. Temperatures can be as high as 50o C compared with the normal range of in-ground temperature of 12o C in cold and temperate climates to 30o C in arid desert climates.

Degradation most commonly occurs from mechanical damage, long-term time dependent degradation caused by stress (creep), deterioration from exposure to ultraviolet light, and chemical or biological interaction with the surrounding environment. Because of varying polymer types, quality, additives and product geometry, each geosynthetic is different in its resistance to aging and attack by different chemical and biological agents. Therefore, each product must be investigated individually.

Typically, polyester products (PET) are susceptible to aging strength reductions due to hydrolysis (water availability) and high temperatures. Hydrolysis and fiber dissolution are accelerated in alkaline regimes, below or near piezometric water levels or in areas of substantial rainfall where surface water percolation or capillary action ensures water availability over most of the year.

Polyolefin products (PP and HDPE) are susceptible to aging strength losses due to oxidation (contact with oxygen) and or high temperatures. The level of oxygen in reinforced fills is a function of soil porosity, ground water location and other factors, and has been found to be slightly less than oxygen levels in the atmosphere (21 percent). Therefore, oxidation of geosynthetics in the ground may proceed at an equal rate than those used above ground.

Oxidation is accelerated by the presence of transition metals (Fe, Cu, Mn, Co, Cr) in the backfill as found in acid sulphate soils, slag fills, other industrial wastes or mine tailings containing transition metals. It should be noted that the resistance of polyolefin geosynthetics to oxidation is primarily a function of the proprietary antioxidant package added to the base resin, which differs for each product brand, even when formulated with the same base resin. Recommended limits of electrochemical properties for backfills when using geosynthetic reinforcements are illustrated in Table 11.2

Table 11.2 Recommended Limits of Electrochemical Properties for Backfills When Using Geosynthetic Reinforcements (Elias, 2001)

Base Polymer	Property Criteria
Polyester (PET)	pH >3<9
Polyolefin (PP & HDPE)	pH >3

Geotextiles vs. Geogrids (Tensar Sierra Slope Retention Systems)

Although biased, Tensar emphasizes that their product, HDPE Geogrids, are superior to PET geotextiles in RSS for the following reasons:

1. Geotextiles have inherent internal slack (mechanical crimp) which requires excessive pre-tensioning in actual installations. Furthermore, geotextiles require considerable srain to achieve peak strength; a condition which may not be tolerable for RSS.
2. Seaming and overlap connections create zones of weakness and deformation.
3. Geotextiles can clog diminishing permeability and inside slope drainage.
4. Geotextiles are susceptible to degradation which decreases the strength available to reinforce the structure. Table 11.3 shows aging reduction factors of 1.1 for HDPE geogrids vs. 1.6+ for PET geotextiles.

CONSTRUCTION SEQUENCE RSS CONSTRUCTION (Elias, 2001)

Reinforced soil systems consist of planar reinforcements arranged in nearly horizontal planes in the backfill to resist outward movement of the reinforced fill mass. Facing treatments ranging from vegetation to flexible armor systems are applied to prevent unraveling and sloughing of the face. Consideration must be given to the choice of slope facing for RSS structures, and may be controlled by climatic and regional factors. For structures of less than 10 m (33 ft) height with slopes of 1:1 or flatter, a vegetative "green slope" can be usually

Table 11.3 Aging Reduction Factors for PET and HDPE Geosynthetics (from Elias, 2000)

Products	Aging Reduction Factor, RF_D - 100 years				
	pH < 3	3 < pH < 5	5 < pH < 8	8 < pH < 9	pH > 9
PET Geotextiles Mn < 200,0000, 40 < GEG < 50	Not Recommended *	2.0	1.6	2.0	Not Recommended*
PET Coated Geogrids Mn > 25,000, CEG < 30	Not Recommended *	1.3	1.15	1.3	Not Recommended *
Polypropylene & HDPE Geogrids	1.1	1.1	1.1	1.1	1.1
Compiled from, "Corrosion/Degradation of Soil Reinforcements for M.S.E. Walls and Reinforced Soil Slopes" (FHWA-NHI-00044) Use of materials outside the indicated pH or molecular property range requires product specific testing.					

constructed using an erosion control mat or mesh and local grasses. Where vegetation cannot be successfully established and/or significant run-off may occur, armored slopes using natural or manufactured materials may be the only choice to reduce future maintenance.

The construction of RSS embankments is considerably simpler and consists of many of the elements outlined for MSEW construction. They are summarized as follows:

1. Site preparation.
2. Construct subsurface drainage (if indicated).
3. Place reinforcement layer.
4. Place and compact backfill on reinforcement.
5. Construct face or install secondary reinforcement and erosion control facing.
6. Place additional reinforcement and backfill.
7. Construct surface drainage features.

Facing Construction

Slope facing requirements will depend on soil type, slope angle and the reinforcement spacing as shown in Table 11.4. If slope facing is required to prevent sloughing (i.e., slope angle β is greater than ϕ_{soil}) or erosion, several options are available. Sufficient reinforcement lengths could be provided for wrapped faced structures. A face wrap may not be required for slopes up to 1H:1V. 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. For wrapped or no wrap construction, the reinforcement should be maintained at close spacing (i.e., every lift or every other lift but no greater than 400 mm (16 inches)). For armored, hard faced systems the maximum spacing should be no greater than 800 mm (32 inches). A positive frictional or mechanical connection should be provided between the reinforcement and armored type facing systems.

Table 11.4 RSS Slope Facing Options

Slope Face Angle and Soil Type	Type of Facing			
	When Geosynthetic is Not Wrapped at Face		When Geosynthetic is Wrapped at Face	
	Vegetated Face[1]	Hard Facing[2]	Vegetated Face[1]	Hard Facing[2]
> 50° (> ~0.9H: IV) All Soil Types	Not Recommended	Gabions	Sod Permanent Erosion Blanket w/seed	Wire Baskets Stone Shotcrete
35° to 50° (~1.4H:IV to 0.9H:IV) Clean Sands (SP)[3] Rounded Gravel (GP)	Not Recommended	Gabions Soil-Cement	Sod Permanent Erosion Blanket w/seed	Wire Baskets Stone Shotcrete
35° to 50° (~1.4H:IV to 0.9H:IV) Silts (ML) Sandy Silts (ML)	Bioreinforcement Drainage Composites[4]	Gabions Soil-Cement Stone Veneer	Sod Permanenet Erosion Blanket w/seed	Wire Baskets Stone Shotcrete
35° to 50° (~1.4H:IV to 0.9H:IV) 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	Seosynthetic Wrap Not Needed
25° to 35° (~2H:IV to 1.4H:IV) All Soil Types	Temporary Erosion Blanket w/Seed or Sod Permanent Erosion Mat w/Seed or Sod	Hard Facing Not Needed	Geosynthetic Wrap Not Needed	Geosynthetic Wrap Not Needed

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

For geogrids, a fine mesh screen or geotextile may be required at the face to retain back-fill materials. Slopes steeper than approximately 1:1 typically require facing support during construction.

RSS COST ESTIMATES (Elias, 2001, Appendix C)

Cost estimates for reinforced slope systems are generally per square meter of vertical face. Table 11.5 provides cost estimates. High RSS structures have higher reinforcement, but lower backfill costs than lower slopes. Recent bid prices suggest costs ranging from $110 to $260 per m^2 for higher slopes. Lower height slopes (10- to 15- m) have reported bid prices of approximately $170 m^2.

Table 11.5 Estimated Geosynthetic Costs

Geosynthetic	Material Cost ($/m^2)
Filtration Geotextiles	1.25 – 1.75
Erosion Control Mats	3.50 – 6.00
Geotextile Embankment Reinforcement	2.50 – 12.00
Geogrid/Goetextile Wall or Slope Reinforcement per 15 kN/m $T_{allowable}$	1.50 – 3.50

DESIGN OF REINFORCED SOIL SLOPES (Christopher, 1990)

Currently, RSS are designed; (1) simplified design charts or (2) via computer programs that are modified versions of classical slope stability program. In both cases, the reinforcement provides additional tensile force as a resisting moment for designing *external* stability. However, the *internal* stability of frictional pullout and tensile breakage must be satisfied to develop this additional tensile force. Most RRS are limited to 70°.

Performance Requirements (Christopher, 1990)

Recommended minimum safety factors (FS) are:

1. Sliding FS > 1.5
2. Deep seated circular failure FS > 1.3
3. Dynamic loaded slopes FS > 1.1

Simplified Design Charts (Jewell, 1980, and Gary Schmertmann, 1987)

Several design charts are available for cohesionless soils (c = 0). The charts use a two wedge analysis, and incorporate a coefficient of earth pressure K to find the required external tensile force for stability (Rimoldi, 1994).

Referring to Figure 11.3

$$U_{h1} = - U_{h2}$$

$$T_1 = \frac{W_1 - (\tan\theta_1 - \tan\phi') - \left(c\frac{l_1}{\cos\theta_1}\right) + U_1\frac{\tan\phi'}{\cos\theta_1}}{1 + \tan\theta_1 \tan\phi'} + U_{h1}$$

and

$$T_2 = \frac{W_2 - (\tan\theta_2 - \tan\phi') - \left(c\frac{l_2}{\cos\theta_2}\right) + U_2\frac{\tan\phi'}{\cos\theta_2}}{1 + \tan\theta_2 \tan\phi'} + U_{h2}$$

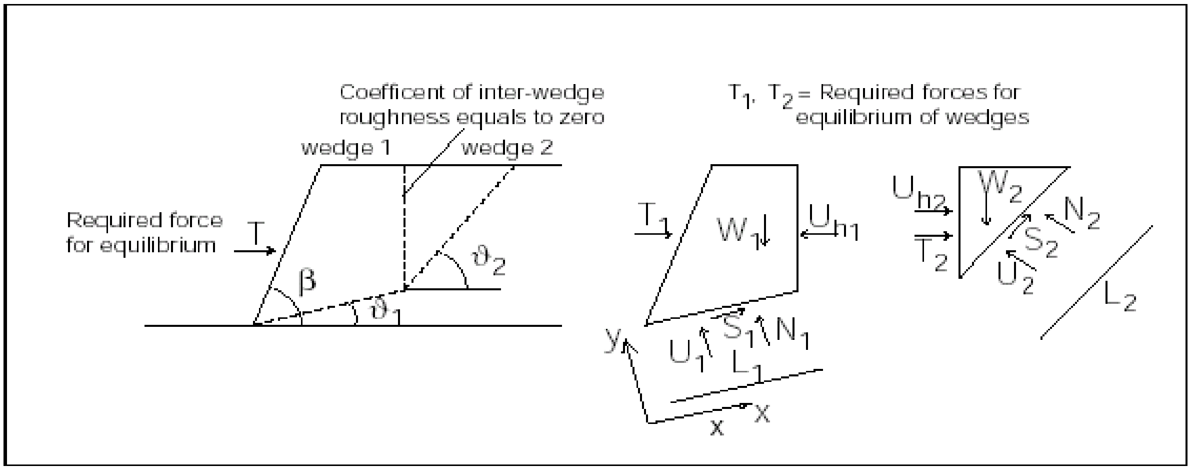


Figure 11.3 Diagram of Two Part Wedge Analysis (from Jewell, and Tensar, 1990)

$$T = T_1 + T_2$$

with c', and ϕ = effective cohesion and friction angle

By combining the above equations: $T = W \bullet f_1(\phi, \theta_1, \theta_2) - c \bullet f_2(\phi, \theta_1, \theta_2) + u \bullet f_3(\phi, \theta_1, \theta_2)$

with $f_{1,2,3}$ = functions of the terms in parentheses.

T can now be expressed as an *earth pressure coefficient*, K, using a Rankine – like active earth pressure equation as:

$T = ½ K \gamma H^2$ with K = coefficient of earth pressure in terms of β, ϕ

γ = unit weight of soil

H = slope height

Also normalizing by γ: $\frac{T}{\gamma} = \frac{W}{\gamma} \bullet f_1(\phi, \theta_1, \theta_2) - \frac{c}{\gamma} \bullet f_2(\phi, \theta_1, \theta_2) + \frac{u}{\gamma} \bullet f_3(\phi, \theta_1, \theta_2)$

But: W/γ = Volume (V) = Area (A) • (unit width = 1); where V = volume of 2 wedges and A = area of 2 wedges.

Using the pore pressure coefficient, $r_u = u / (\gamma H)$ and c = 0 then:

$$\frac{T}{\gamma} = A \bullet f_1(\phi, \theta_1, \theta_2) + r_u \bullet f_3(\phi, \theta_1, \theta_2)$$

Rearranging the earth pressure equation: $K = \frac{2}{H^2} \frac{T}{\gamma}$, and setting H = 1 (unity) results in:

$K = 2 \bullet \frac{T}{\gamma}$, which allows one to find K for a slope of unit height and T / γ

By systematically varying β and ϕ, setting c = 0, and assuming r_u values, design charts (Figure 11.4) can be generated.

Steep Reinforced Slope Design Charts (Jewell, 1991)

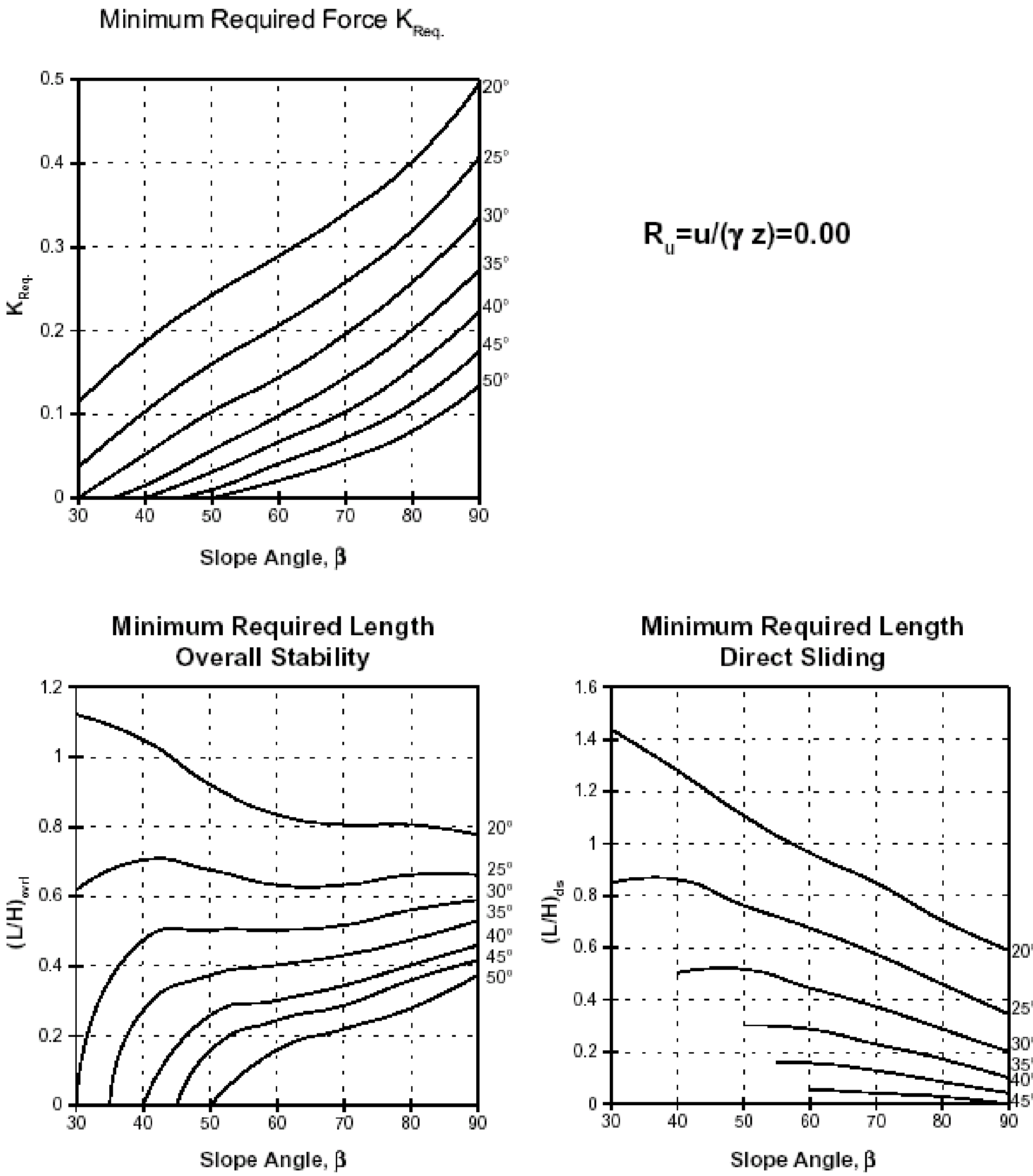


Figure 11.4a Design Chart – Steep Reinforced Slopes R_u = 0.0

Steep Reinforced Slope Design Charts (Jewell, 1991)

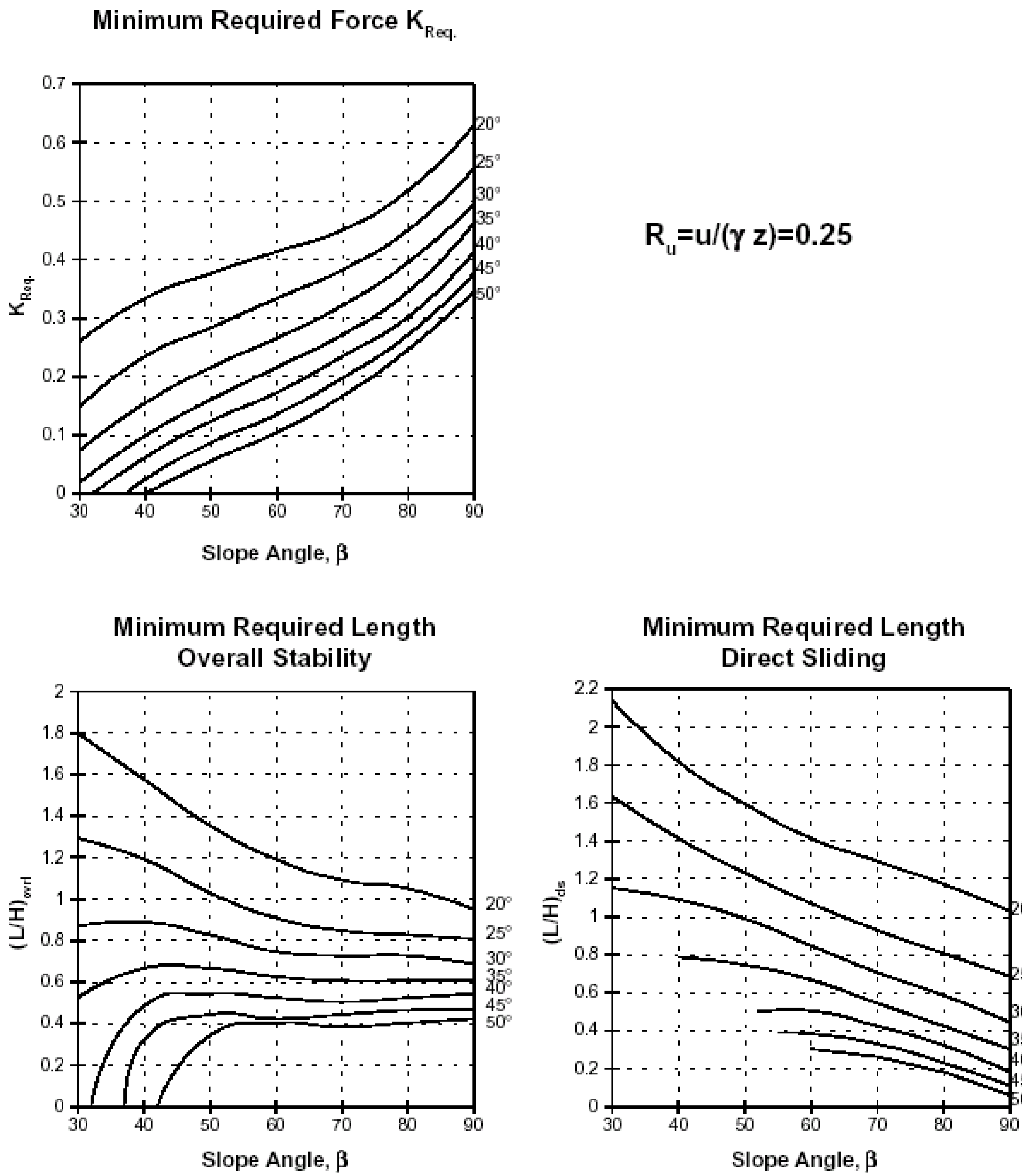


Figure 11.4b Design Chart – Steep Reinforced Slopes R_u = 0.25

Steep Reinforced Slope Design Charts (Jewell, 1991)

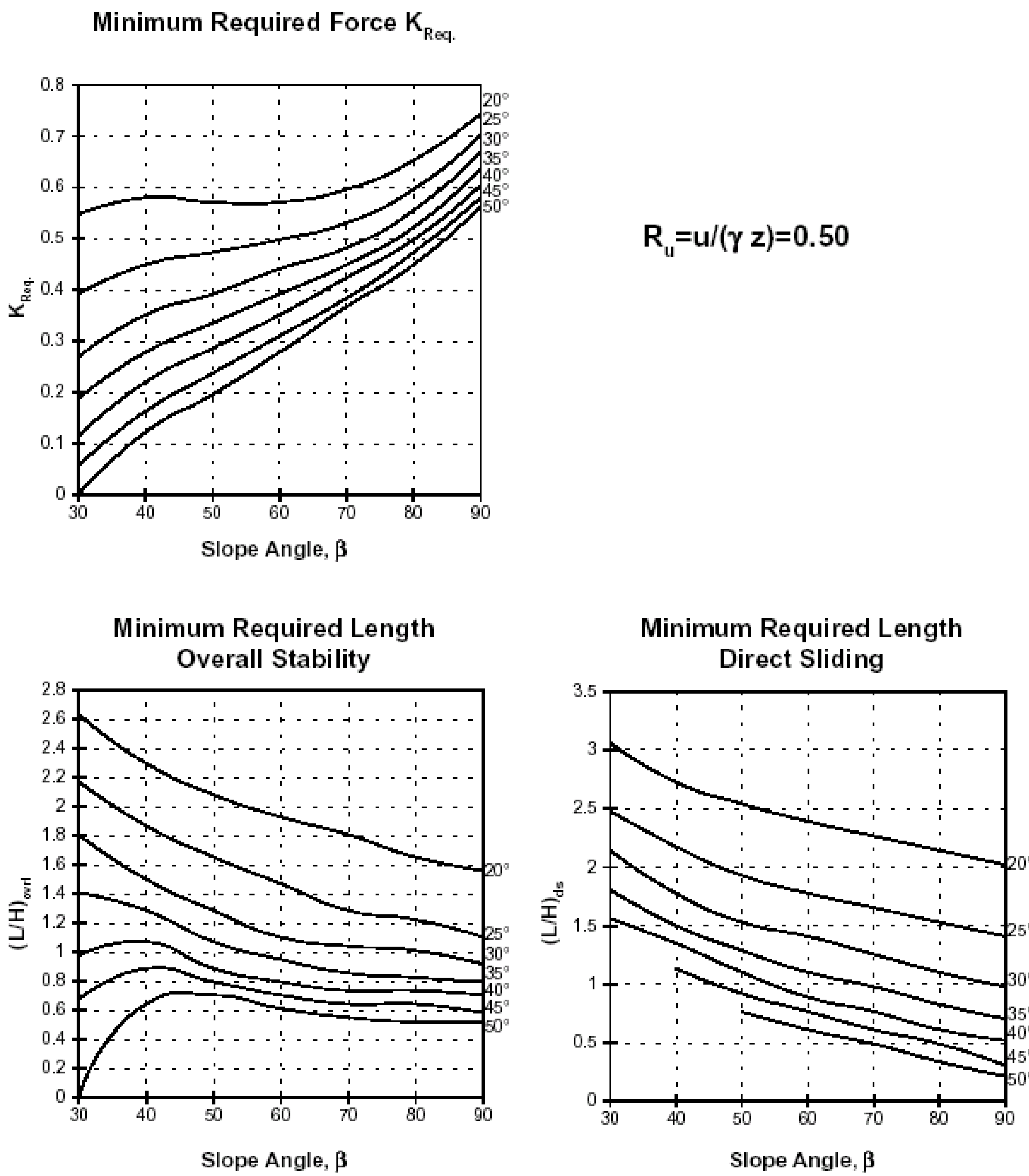


Figure 11.4c Design Chart – Steep Reinforced Slopes R_u = 0.50

These charts are applicable for steep uniform cohesionless (c = 0) slopes with slope angles, from 30° to 90° and horizontal crests. Uniform surcharges can be accommodated by use of an equivalent height. Anchorage lengths for direct sliding or pullout can be estimated from (L/H) ratios given in the charts.

Performance Requirements

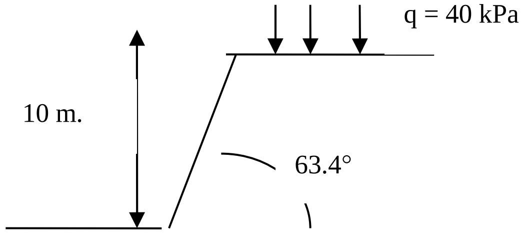
Design performance requirements are (Elias, 2001):

1. External stability
   a. Sliding: F.S. ≥ 1.3.
   b. Deep seated (overall stability): F.S. ≥ 1.3.
   c. Local bearing failure (lateral squeeze): F.S. ≥ 1.3.
   d. Dynamic loading: F.S. ≥ 1.1.
   e. Internal slope stability: F.S. ≥ 1.3.
2. Evaluate design parameters for the reinforcement

   Allowable geosynthetic strength, $T_{allowable}$ = ultimate strength (T_{ULT}) ÷ reduction factor (RF) for creep, installation damage and durability: For granular backfill, RF = 7, may be conservatively used for preliminary design. However, there is a significant cost advantage in obtaining lower RF from test data supplied by the manufacture and/or from agency evaluation
3. Pullout Resistance
   a. F.S. = 1.5 for granular soils.
   b. Use F.S. = 2 for cohesive soils.
   c. Minimum anchorage length, Le, = 1 m (3 ft).

Example Problem - RSS Design Charts

Select a geogrid, and design the geogrid spacing for a 10m high, 2V:1H slope supporting a uniform surcharge of 40 kPa with a factor of safety = 1.5. The slope consists of uniform sand ϕ = 29°, and γ = 20 kN/m^3. Assume r_u = 0.0



1. Calculate modified slope height for surcharge: $H' = H + q/\gamma = 10 + 40/20 = 12$ m.
2. Using FS = 1.5, find factored $\phi = \tan^{-1}(\tan \phi/FS) = \tan^{-1}(29°/1.5) = 20°$.
3. Determine K_{rqd} force from chart. K = 0.3.
4. Calculate *Total* horizontal grid force required. $T = \frac{\gamma H^2}{2} \bullet K = ½\ (20)\ (12)^2\ (0.3) = 432$ kN/m.
5. Examine *Allowable* grid strengths. (www.tensarcorp.com)

Type	Allowable Sand, silt, clay kN/m (lbs/ft)	Allowable Sand (WG) kN/m (lbs/ft)	Allowable Crushed Agg. kN/m (lbs/ft)
UX1100HS	23.7 (1620)	23.0 (1580)	22.8 (1550)
UX1400HS	29.2 (2800)	28.4 (1950)	27.9 (1910)
UX1500HS	45.2 (3100)	44.0 (3010)	43.2 (2960)
UX1600HS	59.9 (4100)	58.2 (3990)	57.2 (3920)
UX1700HS	75.1 (5140)	73.0 (5000)	71.7 (4910)
UX1800HS	77.8 (5330)	75.7 (5180)	74.3 (5090)

6. Determine the minimum number of geogrids required: $N_{min} = T / T_{allowable}$ and select a reasonable number of geogrid layers for design.

Type	Allowable Sand (WG) kN/m (lbs/ft)	Minimum No. of Geogrids N_{min}
UX1100HS	23.0 (1580)	18.7
UX1400HS	28.4 (1950)	15.2
UX1500HS	44.0 (3010)	9.8
UX1600HS	58.2 (3990)	7.4
UX1700HS	73.0 (5000)	5.9
UX1800HS	75.7 (5180)	5.7

7. Assuming compaction lifts of 0.3m, then H = 10m / 0.3m = 33 compaction lifts. Select UX1500HS with N_{min} = 10 layers.
8. FHWA (Christopher,1990) recommends the following distribution of reinforcement:
   a. For low slopes (H < 6m, 20 ft.) assume a uniform reinforcement distribution.

b. For high slopes (H > 6m, 20 ft), divide the slope into 2 (top and bottom) or 3 (top, middle, bottom) reinforcement zones of equal height and use T_{max} in each zone for reinforcement spacing.

c. For 2 Zones:

i. T_{bottom} = ¾ T

ii. T_{top} = ¼ T

d. For 3 Zones:

i. T_{bottom} = ½ T

ii. T_{middle} = 1/3 T

iii. T_{top} = 1/6 T

e. Since H > 6 m , use 3 zones

i. T_{bottom} = 0.5 (432) = 216 kN/m ÷ 44 = 5 layers

ii. T_{middle} = 0.33 (432) = 144 kN/m ÷ 44 = 3.3 layers

iii. T_{top} = (1/6) (432) = 72 kN/m ÷ 44 = 1.7 layers

Total = 10 layers

f.

Zone	Layer No.	Elevation (m) from bottom
Top Zone Spacing = 3.3m ÷ 1.7 layers = 1.9m spacing	10	9.6
	9	7.7
Middle Zone Spacing = 3.3m ÷ 3.3 layers = 1.0m spacing	8	5.8
	7	4.8
	6	3.8
Bottom Zone Spacing = 3.3m ÷ 5layers = 0.7m spacing	5	3.1
	4	2.4
	3	1.7
	2	1.0
	1	0.3

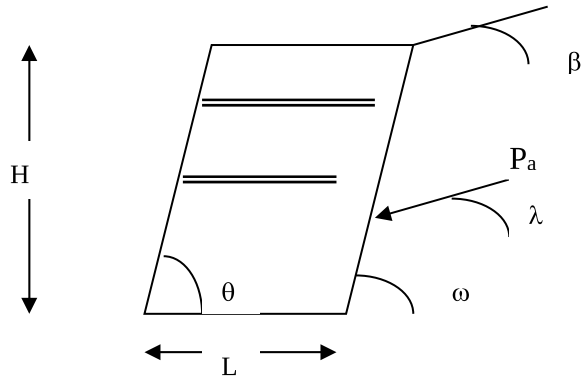
9. Determine geogrid length:

a. From Chart $(L/H)_{ovrl}$ = 0.8, $(L/H)_{ds}$ = 0.9. Use larger

L = (0.9) (12m) = 10.8m

10. Intermediate reinforcement: Use BX1100 as intermediate lifts L = 1-1.5m for erosion control.

11. FHWA (Christopher, 1990) recommends evaluating *external* stability as a sliding block



a. The thrust angle, λ, is assumed parallel to the backfill slope; i.e.,

$\lambda = 90^\circ - \omega + \beta$, except $\lambda \le \phi_{backfill}$

b. FS sliding = Resisting force, F_r / Sliding force, F_{sl}

c. Sliding force $F_{sl} = P_a \cos(\lambda + \omega - 90) = \left[(1/2)\gamma_b H^2 K_a - 2c_b H\sqrt{K_a}\right]\cos(\lambda + \omega - 90)$

d. $$K_a = \left[\frac{\dfrac{\sin(\omega-\phi)}{\sin\theta}}{\sqrt{\sin(\omega+\lambda)} + \sqrt{\dfrac{\sin(\phi+\lambda)\sin(\phi-\beta)}{\sin(\theta-\beta)}}}\right]^2$$; where β = 0°, Lb = LT ∴ θ = ω =

63.4° and λ = 90° - 63.4° + 0° = 26.6° ≤ 29°

$$K_a = \left[\frac{\dfrac{\sin(63.4-29)}{\sin 63.4}}{\sqrt{\sin(63.4+26.6)} + \sqrt{\dfrac{\sin(29+26.6)\sin(29-0)}{\sin(63.4-0)}}}\right]^2 = 0.143$$

e. Horizontal sliding force, P_{sl} = [(½) (20) (12)^2 (0.143)]cos(26.6+63.4-90) = 206kN

f. Calculate the resisting force, P_R = W tan ϕ

   i. W = (10m)(10.8m)(20kN/m^3) = 2,160

   ii. P_R = 2,160 tan 29° = 1,197 kN

g. Factor of Safety (sliding) = P_R / P_{sl} = 1197/206 = 5.81 > 1.5

Computer Program RSS (Reinforced Slope Stability) (Geocomp Corporation 1995)

RSS may be downloaded free of charge from the FHWA Geotechnical Information Center at http://www.fhwa.dot.gov/bridge/geosoft.htm or a disk copy may be purchased from the Center for Microcomputers in Transportation (McTrans) at www.mctrans.ce.ufl.edu. The program is supported by FHWA for all state and federal agencies. For private sector users and others, a supported licensed version is available from the developer GEOCOMP through their web page at www.geocomp.com/software.htm. A windows version of the reinforced soil slope program, ReSSA2.0 is available from
http://www.geoprograms.com/diablowebalternate/ressaorder.htm.

The purpose of RSS is to give the design engineer a convenient tool for; (1) analyzing the stability of an existing slope, (2) determining the changes is stability obtained by adding reinforcement, and (3) finding the reinforcement spacing and strength to obtain a required level of safety. RSS has its foundation in the STABL computer program.

The simplified Bishop circular arc method of slices is used for analyses involving reinforcement. The simplified Janbu method is used for determining reinforcement length to prevent block sliding.

Theory (Christopher, 1990)

The methodology used by RSS involves:

1. Evaluating the FS of the unreinforced slope shown in Figure 11.5, where:

$$FS_U = \frac{\text{Resisting moment } (M_R)}{\text{Driving moment } (M_D)} = \frac{\int_0^L \tau_f \bullet R \bullet dL}{(Wx + \Delta qd)}$$

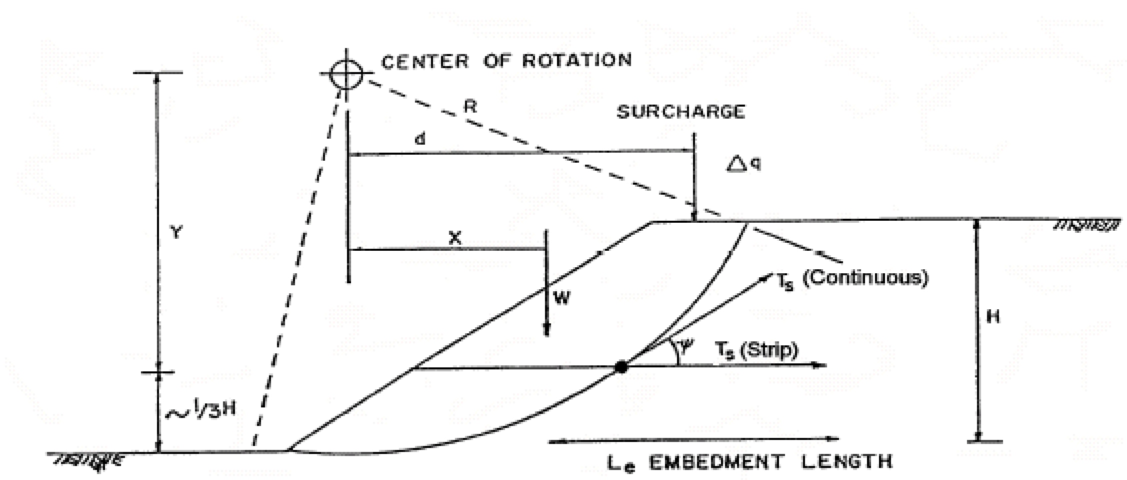


Figure 11.5 Circular Arc Design Assumptions for Reinforced Slope

2. Reconsidering the reinforced slope, where the reinforcement is considered as an addition to the resisting moment. Hence, the reinforced FS is: $FS_R = \frac{M_R + T_S \bullet D}{M_D}$. For extensible reinforcement ;i.e., geogrids, D equals the radius of the trial circle.

3. Determining the quantity of reinforcement required to obtain a specified FS, the preceding equation is rearranged to give: $T_S = (FS_R - FS_U) \bullet \frac{M_D}{D}$

4. Analyzing various trial circles, computing T_s until the maximum T_s required is determined as T_{max}

5. Proportioning the reinforcement spacing within the slope.
   a. For slopes > 20 ft (6 m) T_{max} is apportioned into 3 zones:
      i. T_{bottom} = 0.500 T_{max}
      ii. T_{middle} = 0.333 T_{max}
      iii. T_{top} = 0.167 T_{max}
   b. For slopes < 20 ft (6 m), T_{max} is proportioned uniformly throughout the slope.

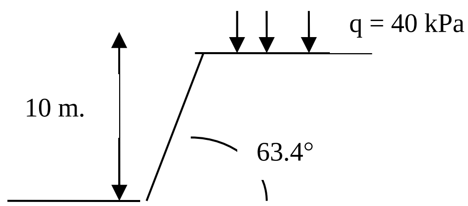
6. If the more common case where the reinforcement strength $T_{allowable}$ is known, the spacing can be determined proportioning within the zones. If the reinforcing spacing is known; i.e., designated lift thickness, the reinforcement strength can be determined from; $T_s = T_{max} \bullet \frac{D}{\sum d_i}$, where d_i = lift thickness.

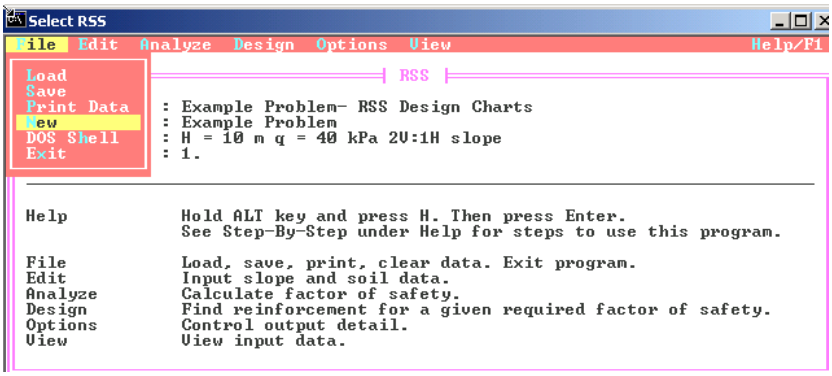
7. Knowing the reinforcement strength and spacing for the most critical circle, the length of reinforcement can be determined as the horizontal distance from the slope to the sliding surface plus the required length of embedment. The required length of embedment is calculated as: $L_e = \frac{T_a \bullet FS_{pullout}}{2 \bullet F^* \bullet \alpha \bullet \sigma_v'}$; where:
   a. Le = Required length of embedment
   b. T_a = Allowable design strength of reinforcement
   c. $FS_{pullout}$ = FS against pullout, usually 1.5
   d. F* = pullout friction factor, for estimate use 0.6 tanϕ
   e. α = correction factor to account for shear stress mobilization that is less than peak along the embedded length of the reinforcement; for estimate use 0.6
   f. σ_v' = effective vertical stress at soil – reinforcement interface.

8. Sliding block analyses using the simplified Janbu method are performed next to determine the reinforcement length required to prevent wedge type sliding failures. The deepest sliding wedge with a FS equal to the required FS for sliding is found. The reinforcement length in the critical circle is lengthened, if necessary, to intersect the backplane of the critical sliding block for the bottom, middle, and top one-third zones of the slope.

Example Problem- RSS Computer Program

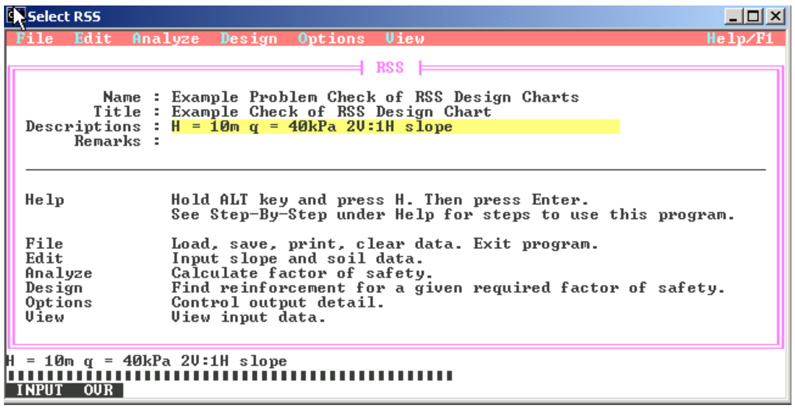
Select a geogrid, and design the geogrid spacing for a 10 m high, 2V:1H slope supporting a uniform surcharge of 40 kPa with a factor of safety = 1.5. The slope consists of uniform sand ϕ = 29°, and γ = 20 kN/m^3. Assume r_u = 0.0



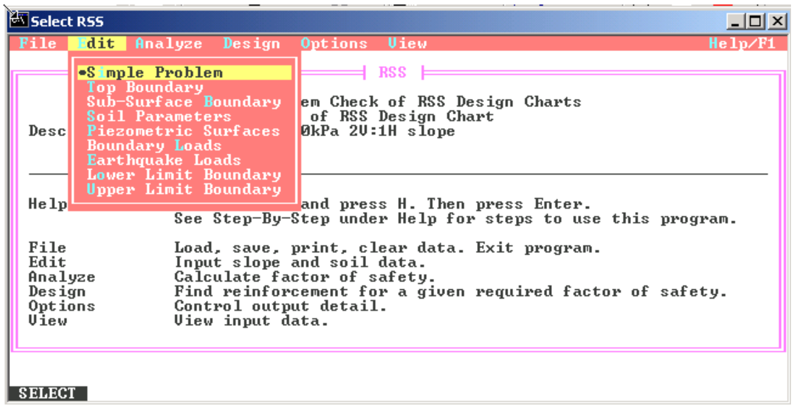


Opening screen

Alt F and select "new file"



Type title of new file

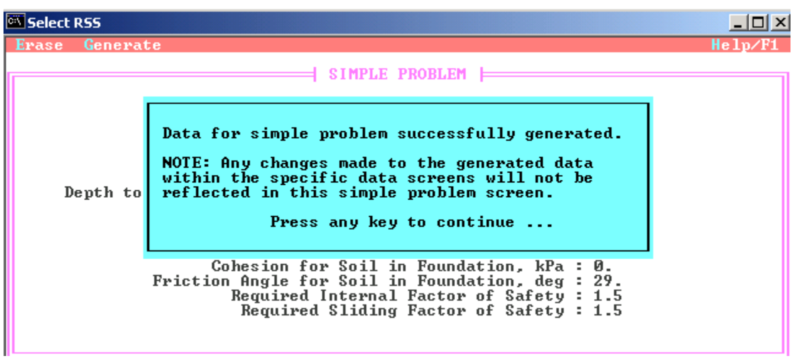


Alt E selects edit screen

Move the cursor to the option screen:

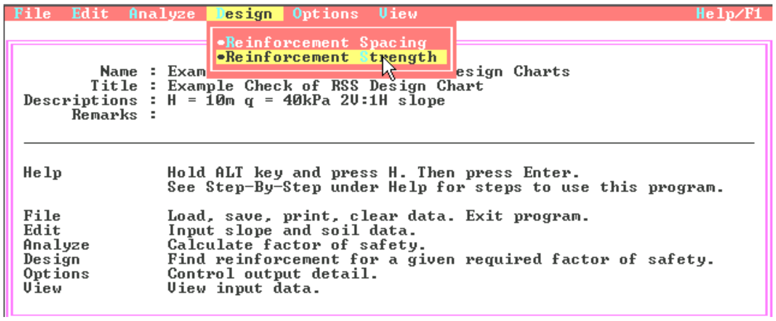
Simple Problem and press

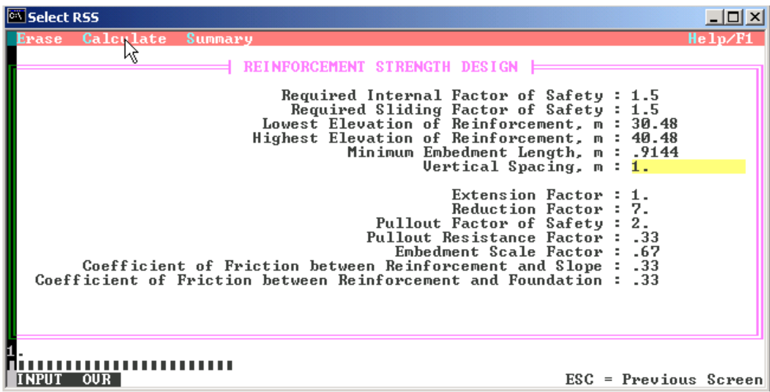
Enter ↵



Enter in the input values for parameters requested on the menu as shown

If an explanation of any of the input parameters is required, move the cursor to that item and press the F1 key. Hold the Alt key and press G to generate all information required for geometry, soil properties and water data. If you exam one the other various Edits submenu items (e.g., Top Boundary), you will see that all of the required data have been automatically inserted. Hold the Alt key and press M to return to the main menu or press the Esc key For a view of the input, press Alt V.



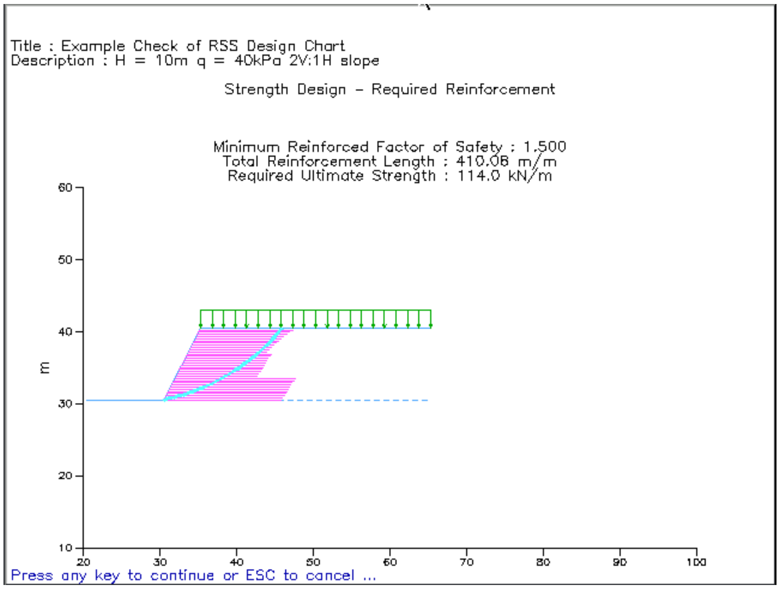


Hold the Alt key and press D to select the design option. This problem requires a determination of the required strength of the reinforcement for a fixed vertical spacing. Therefore, the Reinforcement Strength option is selected (by moving the cursor to that option and pressing enter) and the vertical spacing of 0.3m (1 ft) is used (Note: If the reinforcement strength was known, i.e., from a preapproved products list, the program could also be used to calculate the required spacing.)

Most of the other listed information is already set to match the inputted information and preset default values. For this example, continuous geogrid reinforcement will be used so 1.0 is entered for the case of full extension. A recommended reduction factor, R_f , of 7 is the default value, default values are also used for interaction parameters. Help F1 explains the rationale for these default values.

The design analysis can now be performed. Hold the Alt key and press C to calculate. A box will pop up asking for the Output File Name. This file is where the detailed results for the analysis is to be written.

A graph will appear on the screen showing each trial circle as it is analyzed. When this part finishes, press any key to continue the analysis. A new graph shows the location of reinforcement required for the most critical circle. Press any key to continue. The next graph shows each sliding block as it is analyzed. Press any key to continue and see the location of reinforcement required for the most critical circle modified in the bottom third for the location of the critical sliding block. This process is repeated for sliding blocks in the middle and top thirds of the slope. Press any key to continue and see trial surfaces displayed again as they are analyzed for adequate reinforcement. Press any key one more time to see the final reinforcement spacing and length required. The final graph and summary output screens appear as follow:



RSS Output

The following is an abbreviated edited version of the out put file

```
****************************************************************************
*****                           R S S                                  *****
*****                 Reinforced Slope Stability                       *****
*****                                                                  *****
*****            (c)1992-1996 by GEOCOMP Corp, Concord, MA            *****
*****          licensed to FHWA for distribution by FHWA only          *****
****************************************************************************

         File :
         Date : Tue 02-24-:4, 14:26:19
         Name : Example Problem Check of RSS Design Charts
Problem Title : Example Check of RSS Design Chart
  Description : H = 10m q = 40kPa 2V:1H slope

****************************************************************************
*****                    REINFORCEMENT DATA                            *****
****************************************************************************
```

Data for Reinforcement Strength Design

```
        Required Internal Factor of Safety : 1.50
         Required Sliding Factor of Safety : 1.50
        Lowest Elevation for Reinforcement : 30.48 m
       Highest Elevation for Reinforcement : 40.48 m
                  Minimum Embedment Length : 0.91 m
                          Vertical Spacing : 0.30 m

                          Extension Factor : 1.00
                          Reduction Factor : 7.00
                  Pullout Factor of Safety : 2.00
                 Pullout Resistance Factor : 0.33
                     Embedded Scale Factor : 0.67
             Slope Coefficient of Friction : 0.33
        Foundation Coefficient of Friction : 0.33
```

```
*****************************************************************************
*****                              RESULTS                              *****
*****************************************************************************
```

Unreinforced Circular Surface Tmax

```
                           Circle Center X : 24.68 m
                           Circle Center Y : 56.27 m
                             Circle Radius : 26.43 m
                            Surface Height : 10.00 m
                          Factor of Safety : 0.945
                            Driving Moment : 2.559853E+004 kN-m/m
                    Required Reinforcement : 537.6 kN/m
          Bottom Critical Zone Factor of Safety : 1.502
```

```
*****************************************************************************
*****                      REINFORCEMENT DESIGN                         *****
*****************************************************************************
```

Reinforcement Length per Layer

Layer No.	Elevation (m)	Length (m)
1	30.48	15.32
2	30.78	15.35
3	31.09	15.38
4	31.39	15.41
5	31.70	15.45
6	32.00	15.48
7	32.31	15.51
8	32.61	15.55
9	32.92	15.58
10	33.22	15.61
11	33.53	15.65
12	33.83	10.58
13	34.14	10.61
14	34.44	10.64
15	34.75	10.68
16	35.05	10.71
17	35.36	10.74
18	35.66	10.78
19	35.97	10.81
20	36.27	10.84
21	36.58	10.87
22	36.88	10.91
23	37.19	10.17
24	37.49	10.34
25	37.80	10.49
26	38.10	10.64
27	38.40	10.78
28	38.71	10.97
29	39.01	11.17
30	39.32	11.40
31	39.62	11.62
32	39.93	11.87
33	40.23	12.16

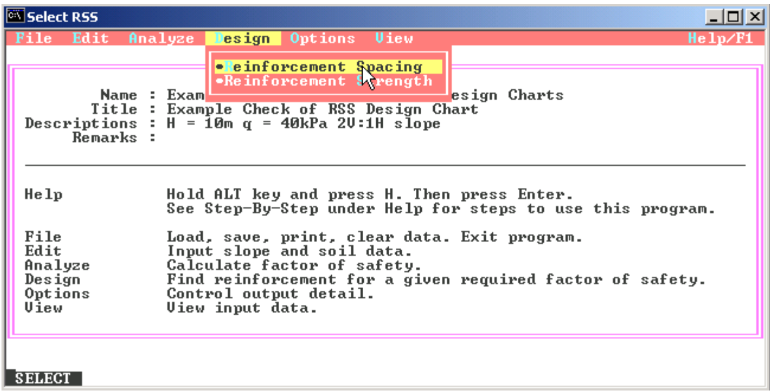
NOTE: The lengths of reinforcement at each height are the minimum lengths of reinforcement necessary to obtain the required factor of safety. For final design, these lengths should be adjusted to values convenient for construction with a given material. If this adjustment results in shorter lengths than computed for some layers, the Reinforcement Analysis option of the program should be used to determine the factor of safety for the adjusted reinforcement pattern.

Minimum Reinforced Factor of Safety : 1.500

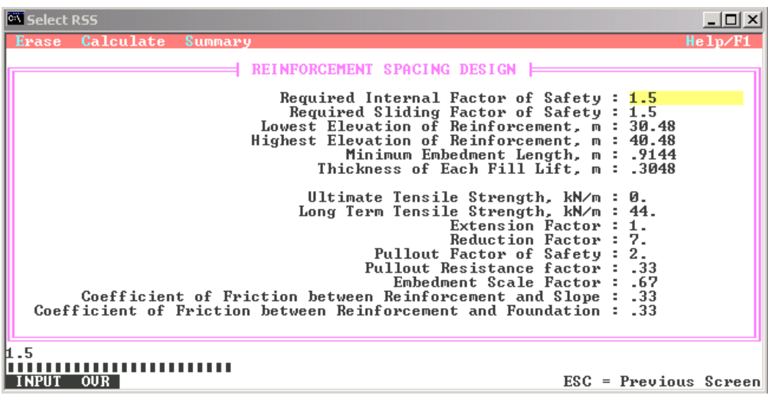
Total Reinforcement Length : 410.08 m/m

Required Ultimate Strength : 114.0 kN/m

```
NOTE: The total required length of reinforcement per unit width of
slope results from the minimum lengths of reinforcement at each
height necessary to obtain the required factor of safety. This
value is provided to help compare reinforcement requirements from
alternate analyses. Since additional reinforcement will be required
for overlaps, face wraps and construction tolerances, this value
should not be used directly to estimate construction quantities.
```



To evaluate spacing based upon a selected geogrid Select “Reinforcement Spacing” under “Design”

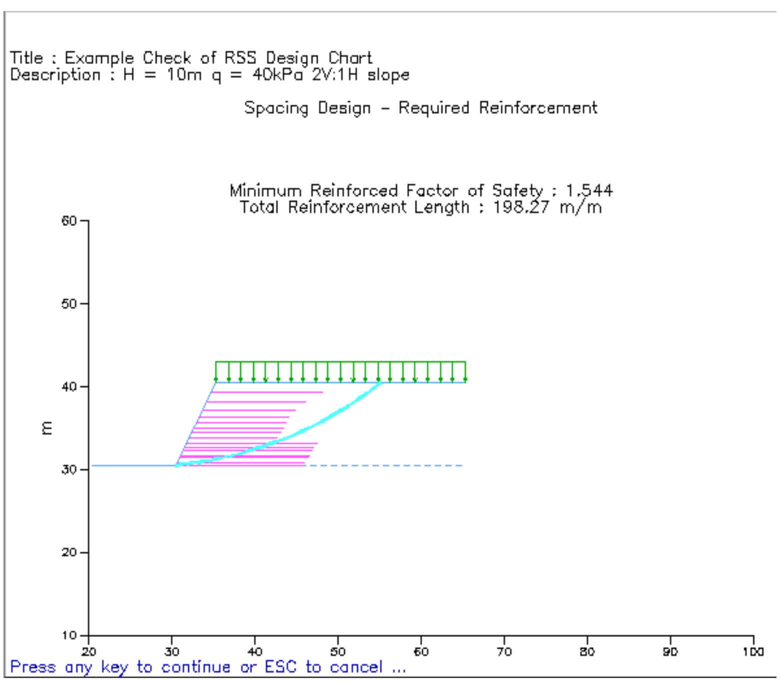


Input geogrid properties

Note: Ult. Tensile Strength = 0, and instead long term tensile strength = 44.0, which is the allowable strength for UX1500 is used.

Because Ult. tensile strength = 0, the reduction factor = 7 is not used.

Alt C performs the calculation



Last screen showing results

RSS Output File

The following is an abbreviated edited version of the output file for "Design – Reinforcement Spacing"

```
 ****************************************************************************
*****                              R S S                               *****
*****                   Reinforced Slope Stability                     *****
*****                                                                  *****
*****              (c)1992-1996 by GEOCOMP Corp, Concord, MA           *****
*****            licensed to FHWA for distribution by FHWA only        *****

         File :
         Date : Wed 02-25-:4, 08:40:17
         Name : Example Problem Check of RSS Design Charts
Problem Title : Example Check of RSS Design Chart
  Description : H = 10m q = 40kPa 2V:1H slope
      Remarks :



****************************************************************************
*****                           INPUT DATA                             *****

Data for Generating Simple Problem

      Note: The following data reflect the data used by Simple Problem
      to automatically generate a data file. Changes made by editing
      that data are not reflected in the Simple Problem data.

              X-Coordinate for Toe of Slope : 30.48 m
              Y-Coordinate for Toe of Slope : 30.48 m
                            Height of Slope : 10.00 m
                             Angle of Slope : 64.3 deg
                 Angle Above Crest of Slope : 0.0 deg
             Surcharge Above Crest of Slope : 40.0 kPa
         Depth to Water from Crest of Slope : 0.00 m
               Unit Weight of Soil in Slope : 20.00 kN/m^3
                 Cohesion for Soil in Slope : 0.00 kPa
           Friction Angle for Soil in Slope : 29.0 deg
          Unit Weight of Soil in Foundation : 20.00 kN/m^3
```

```
        Cohesion for Soil in Foundation : 0.00 kPa
  Friction Angle for Soil in Foundation : 29.0 deg
     Required Internal Factor of Safety : 1.50
      Required Sliding Factor of Safety : 1.50
```

Soil Parameters

```
      Number of Soil Types : 2
```

Soil Type No.	Total Unit Wt. (kN/m^3)	Saturated Unit Wt. (kN/m^3)	Cohesion Intercept (kPa)	Friction Angle (deg)	Pore Pressure Param.	Pressure Constant (kPa)	Piez. Surface No.
1	20.0	20.0	0.0	29.0	0.00	0.0	0
2	20.0	20.0	0.0	29.0	0.00	0.0	0

Boundary Loads

```
      Number of Loads : 1
```

Load No.	X-Left (m)	X-Right (m)	Intensity (kPa)	Inclination (deg)
1	35.29	65.29	40.0	0.0

```
*****************************************************************************
*****                    TRIAL SURFACE GENERATION                      *****
*****************************************************************************
```

Data for Generating Circular Surfaces

```
             Number of Initiation Points : 5
      Number of Surfaces From Each Point : 5
                   Left Initiation Point : 30.48 m
                  Right Initiation Point : 34.09 m
                  Left Termination Point : 35.29 m
                 Right Termination Point : 56.83 m
                       Minimum Elevation : 0.00 m
                          Segment Length : 1.33 m
                     Positive Angle Limit : 57.87 deg
                     Negative Angle Limit : 0.00 deg
```

```
*****************************************************************************
*****                      REINFORCEMENT DATA                           *****
*****************************************************************************
```

```
Data for Reinforcement Spacing Design

        Required Internal Factor of Safety : 1.50
         Required Sliding Factor of Safety : 1.50
        Lowest Elevation for Reinforcement : 30.48 m
       Highest Elevation for Reinforcement : 40.48 m
                  Minimum Embedment Length : 0.91 m
               Thickness of Each Fill Lift : 0.30 m

                          Tensile Strength : 0.00 kN/m
                Long Term Tensile Strength : 44.00 kN/m
                          Extension Factor : 1.00
                          Reduction Factor : 7.00
                  Pullout Factor of Safety : 2.00
                 Pullout Resistance Factor : 0.33
                     Embedded Scale Factor : 0.67
             Slope Coefficient of Friction : 0.33
        Foundation Coefficient of Friction : 0.33

*****************************************************************************
*****                              RESULTS                              *****
*****************************************************************************
```

Unreinforced Circular Surface with T_{max}

```
                   Circle Center X : 24.68 m
                   Circle Center Y : 56.27 m
                     Circle Radius : 26.43 m
                    Surface Height : 10.00 m
                  Factor of Safety : 0.945
                    Driving Moment : 2.560E+004 kN-m/m
                        Moment Arm : 26.43 m
        Allowable Tensile Strength : 44.0 kN/m
      Total Required Reinforcement : 537.6 kN/m

      Bottom Third
            Required Reinforcement : 268.8 kN
                  Number of Layers : 7
               Theoretical Spacing : 0.48 m

      Middle Third
            Required Reinforcement : 179.2 kN
                  Number of Layers : 5
               Theoretical Spacing : 0.67 m
```

Top Third

Required Reinforcement : 89.6 kN
Number of Layers : 3
Theoretical Spacing : 1.11 m

****

***** REINFORCEMENT DESIGN *****

****

Reinforcement Length per Layer

Layer No.	Elevation (m)	Length (m)
1	30.48	15.32
2	30.78	15.35
3	31.39	15.41
4	31.70	15.45
5	32.31	15.51
6	32.61	15.55
7	33.22	15.61
-----	----------	----------
8	33.83	10.58
9	34.44	10.64
10	35.05	10.71
11	35.66	10.78
12	36.27	10.84
-----	----------	----------
13	37.19	11.14
14	38.10	12.00
15	39.32	13.38

NOTE: The lengths of reinforcement at each height are the minimum lengths of reinforcement necessary to obtain the required factor of safety. For final design, these lengths should be adjusted to values convenient for construction with a given material. If this adjustment results in shorter lengths than computed for some layers, the Reinforcement Analysis option of the program should be used to determine the factor of safety for the adjusted reinforcement pattern.

Minimum Reinforced Factor of Safety : 1.544
Total Reinforcement Length : 198.27 m/m

```
NOTE: The total required length of reinforcement per unit width of
slope results from the minimum lengths of reinforcement at each
height necessary to obtain the required factor of safety. This
value is provided to help compare reinforcement requirements from
alternate analyses. Since additional reinforcement will be re-
quired for overlaps, face wraps and construction tolerances, this
value should not be used directly to estimate construction quanti-
ties.

WARNING: Vertical spacing between some reinforcement layers
exceeds the recommended maximum value of 0.6096m. Add intermediate
reinforcement as necessary to provide local stability of the slope
face. Intermediate reinforcement needs not be as long or as strong
as primary reinforcement. See Thielen and Collin (1993) for assis-
tance.
```

Discussion of RSS Results

Table 11.6 summarizes the results of the two design approaches; (1) Reinforcement Spacing, and (2) Reinforcement Strength, and compares with the design charts.

Table 11.6 Comparison of RSS Example Results

Design Criteria	Unreinforced Slope	Reinforced Slope	
	T_{max}	Number of Layers	Total Length
Reinforced Strength T_{ult} = 114 kN/m	537.6 kN/m	33	410.08 m/m
Reinforced Spacing UX1500 $T_{allowable}$ = 44kN/m	537.6 kN/m	15	198.27 m/m
Design Chart UX1500 $T_{allowable}$ = 44kN/m	432 kN/m	10	108 m/m

As shown, the RSS program suggests a greater T_{max} than the design charts. For the reinforced slopes, the reinforced spacing compares with the design chart as UX 1500 was selected. The RSS program suggests more layers and obviously a greater total length than the charts.

REFERENCES

Christopher, B.R., Safdar, A.G., Giroud, J-P, Juran, I., Mitchell, J.K., Schlosser, F., and Dunnicliff, J. (1990) "Reinforced Soil Structures Vol. 1: Design and Construction," FHWA Publ. No. FHWA-RD-89-043.

Collin, J.G., "Controlling Surficial Stability Problems on Reinforced Steepened Slopes" *Geotechnical Fabrics Report* IFAI, 1996. (refers to table 11.4 facing details)

Elias, V. (2000), “Corrosion/Degradation of Soil Reinforcements for Mechanically Stabilized Earth Walls and Reinforced Soil Slopes” FHWA Publ No. FHWA-NHI-00-044.

Elias, V., Christopher, B.R., Berg, R.R. (2001) “Mechanically Stabilized Earth Walls and Reinforced Soil Slopes Design and Construction Guidelines,” FHWA-NHI-00-043.

Elias, V., Welsh, J., Warren, J., and Lukas, R. (1998) “Ground Improvement Technical Summaries,” FHWA Demonstration Project 116, Publ No. FHWA- SA-98-086

Geocomp Corporation (1995) “RSS-Reference Manual” prepared for FHWA, Geocomp Corp., 10 Craig Road, Acton, MA.

Jewell, R.A. (1991) *Application of revised design charts for steep reinforced slopes*, Geotextiles and Geomembranes, Vol. 10, No. 3, 1991, pp. 203-234.

Mitchell, J.K., and Villet, W. (1987) “Reinforcement of Earth Slopes and Embankments,” NCHRP report 290, Transportation Research Board, Washington, D.C.

Rimoldi, P., Ricciuti, A., Recalcati, P. (1994) “Tenax Design Manual; Steep Reinforced Slopes,” www.tenax.net/geosynthetics/tech_doc/

Schmertmann, G.R. (1987), “Design Charts for Geogrid-Reinforced Slopes,” Geosynthetics Conf. ’87, IFAI, New Orleans, LA.

Tensar Sierra Slope Retention System, www.tensarcorp.com

Thielen, D.L., and Collin, J.G., “Geogrid Reinforcement for Surficial Stability of Slopes,” Proceedings of Geosynthetics ’93 Conference, Vancouver, B.C., March, 1993.

Appendix to Chapter 11 – RSS

TENSAR DESIGN CHARTS

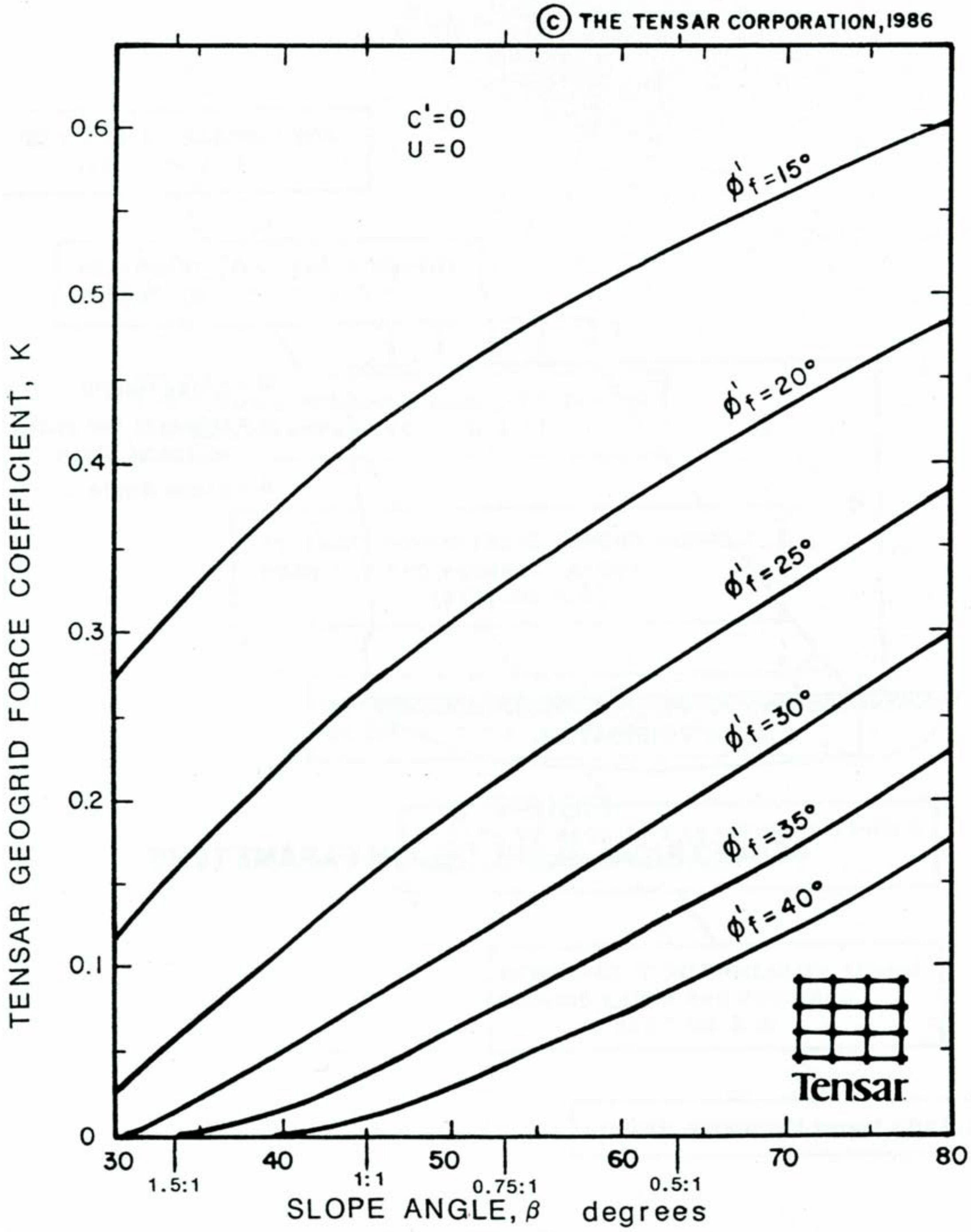


FIGURE 4

TENSAR GEOGRID FORCE COEFFICIENT CHART

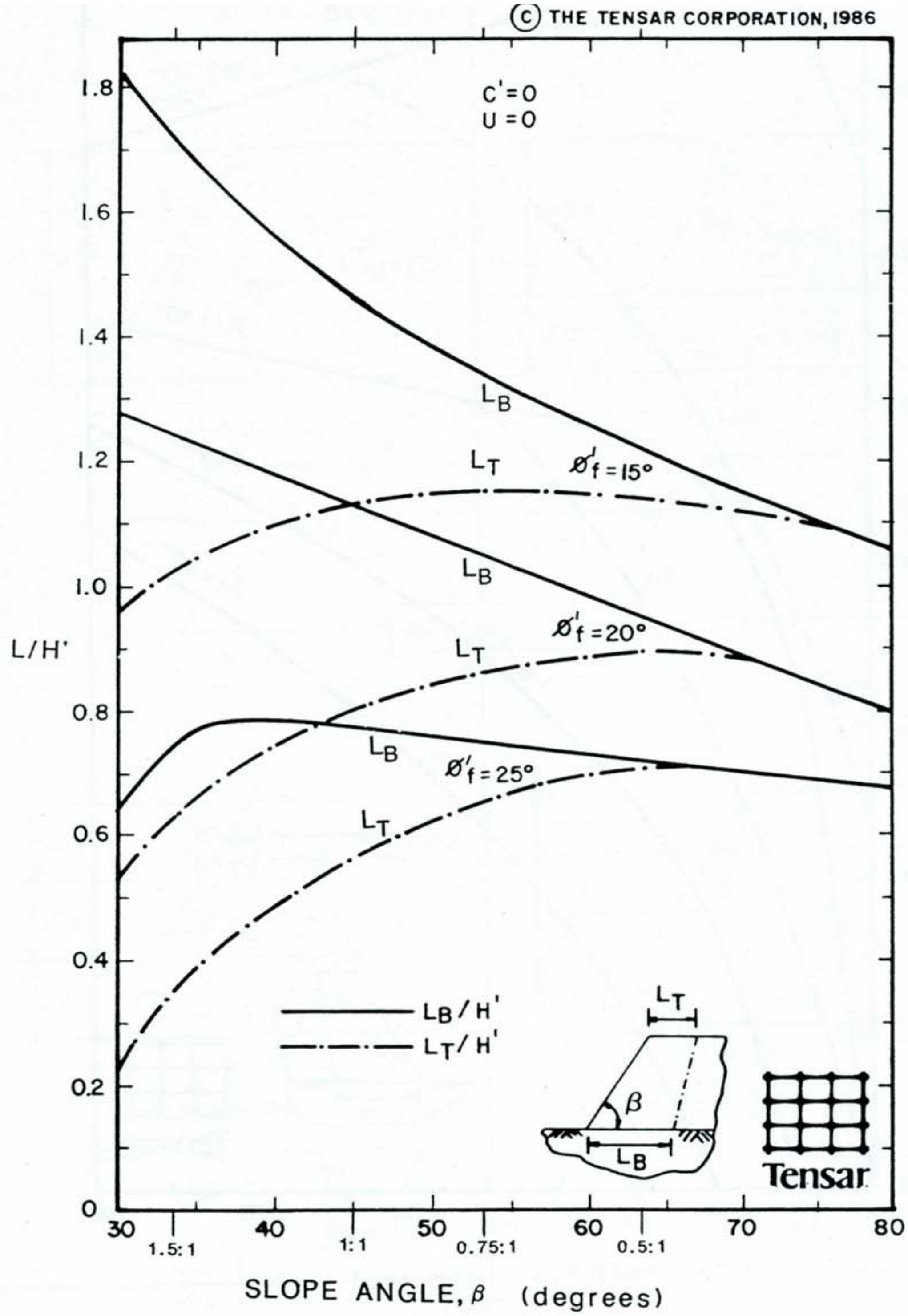


FIGURE 5A

TENSAR GEOGRID LENGTH CHART

$\phi'_f = 15°$ TO $\phi'_f = 25°$

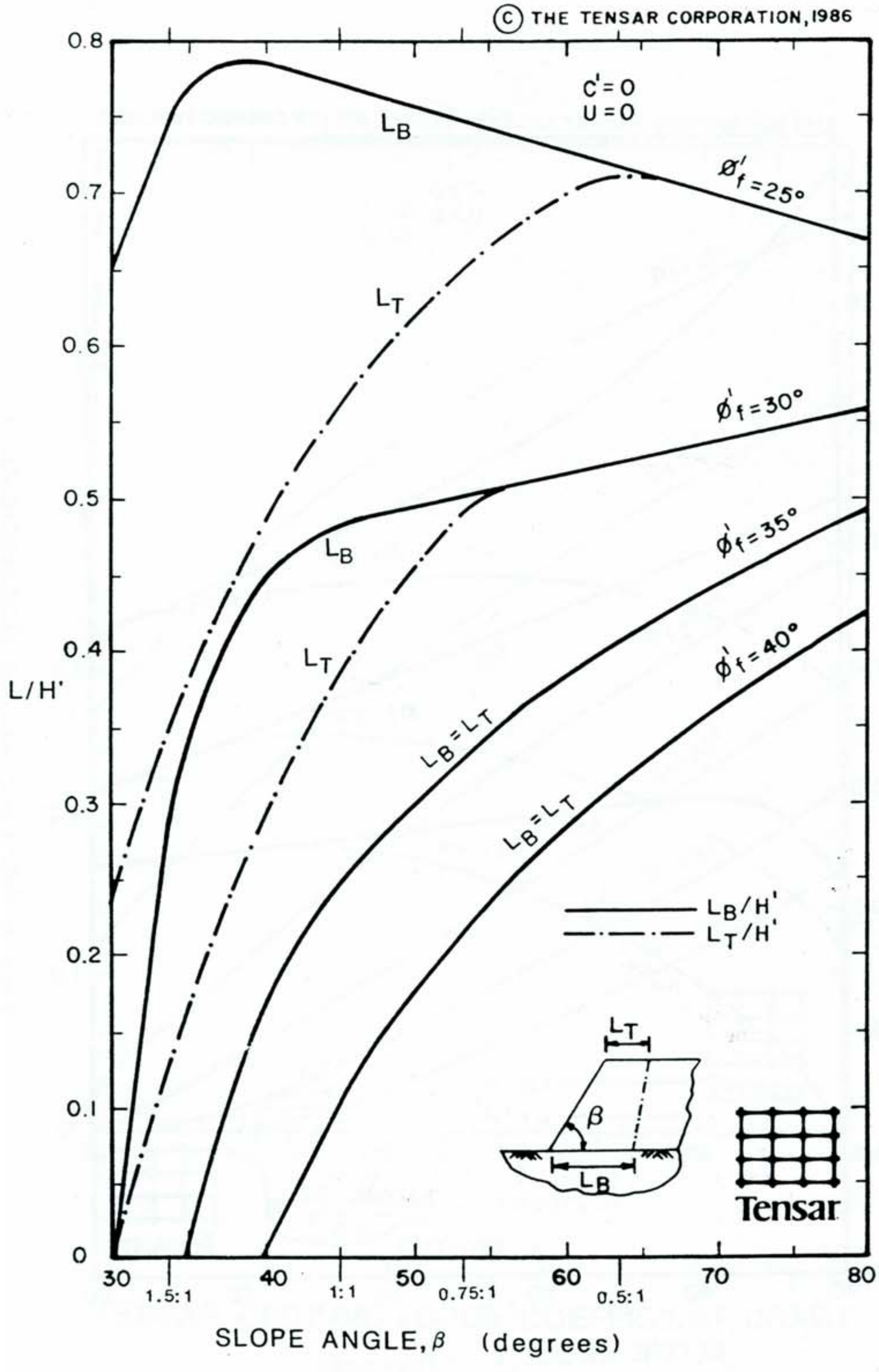


FIGURE 5B

TENSAR GEOGRID LENGTH CHART

$\phi'_f = 25°$ TO $\phi'_f = 40°$

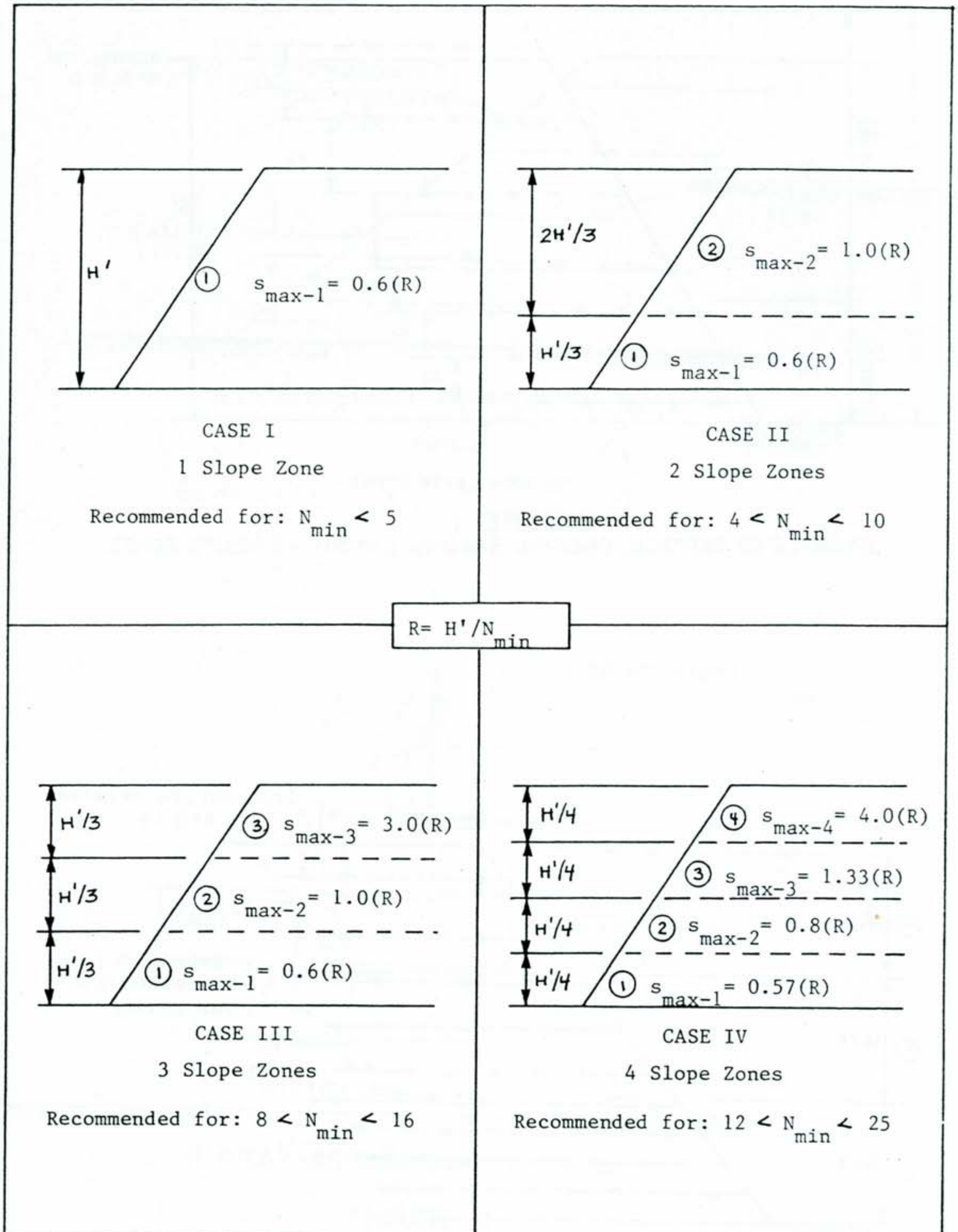


FIGURE 6

Slope Zones for Vertical Spacing of Geogrid Layers

Chapter 12

SOIL NAILING

HISTORY

Soil nailing had its debut in North America as a temporary retaining wall in Vancouver, BC, in the early 1970s. The first documented construction project in the USA to use soil nailing was in Portland, OR, in 1976, for foundation excavation of the Good Samaritan Hospital; the wall height was 45 ft (ENR 1976).

The French contractor Bouygues, in joint venture with specialist contractor Soletranche, is credited with the first recorded application of soil nailing in Europe (1972/73) for an 18-m-high 70° cut slope in Fontainebleau Sand, as part of a railway-widening project near Versailles. Over 25,000 steel bars grouted into pre-drilled holes up to 6-m long stabilized a total of 12,000 m^2 of face (FHWA ,1996).

Based upon the research programs in Germany at the University of Karlsruhe and Bauer (1975-1981) and in France by Clouterre in 1986 the use of soil nail construction has increased popularity in the United States, where it is used primarily for temporary and permanent support of building excavations and for highway projects. The Federal Highway Administration (FHWA) has implemented this technology on highway projects, such as road widening, since the 1980s, and in 1996, the FHWA published guidelines *Manual for the Design and Construction of Soil Nail Walls* for soil nail construction based on the extensive European experience.

There are no proprietary restrictions on the use of the soil nailing concept. However, some specific systems of nails and/or facing are patented. A recently patented (by Soil Nailing Limited, United Kingdom) soil-nailing technique inserts reinforcing nails into the ground by means of a compressed-air "launcher." Various nail installation techniques such as the French-developed "HURPINOISE" and "Jet Nailing" techniques, are patented. One U.S. specialty contractor has taken out a patent on the use of soil nails and tie-backs for repair of existing walls such as corrugated metal bin walls (Schnabel Foundation Co. Patent No. 4, 911, 582 March, 1990).

OVERVIEW OF THE SOIL NAIL PROCESS

Soil nailing is a method of construction that reinforces and strengthens the existing ground by installing closely spaced steel bars "nails," into a slope or existing ground. The process creates a reinforced section that within itself is stable and able to retain the soil behind it. The nails, similar to MSE walls are considered as "passive" because they are not pretensioned (as tieback inclusions are); the nails develop tension as the ground deforms laterally in response to ongoing excavation. In most cases, a temporary or permanent facing is added to retain the soil. Drainage of the site must also be carefully planned and implemented.

A distinct feature of soil nailing is its top-down construction. Excavation occurs in layers of about 6 ft, one layer at a time, from the top of the wall. As each soil layer is excavated, nails are installed and facing is added, then the next layer down is similarly treated. Soil nailing is cost-effective, with savings realized mainly from the ease of construction and the structural benefits of distributing the developed earth pressure loads over a large number of nails. (N. Goldstein, 2001)

Description of the Method (FHWA, 1996)

The unique feature of soil nailing is that the walls are built from the top – down, in small (about 6 ft) successive lifts. Figure 12.1 illustrates the methodology. Essentially, constructions consists of these basic steps:

1. Excavation
2. Drilling hole for nail
3. Nail installation
4. Shotcreting (with or without reinforcement)
5. Repeat steps 1-4
6. Install permanent wall facing (optional).

Depending upon ground conditions, steps 2 and 4 may be reversed. Permanent walls typically add step 6, which consists of placing a permanent wall facing (cast-in-place concrete, or precast facade) over the initial shotcrete layer.

A more expanded description of the process follows: (FHWA, 1996)

A. Excavate Initial Cut

Before commencing excavation, it is necessary to ensure that all surface water will be controlled during the construction process. This is usually done by the use of collector trenches to intercept and divert surface water before it can impact the construction operations. The initial cut is excavated to a depth slightly below the first row of nails, typically about 1 to 2 m depending on the ability of the soil to stand unsupported for a minimum period of 24 to 48 hours. Where face stability is problematical for these periods of time, a stabilizing berm can be left in place until the nail has been installed and final trimming then takes place just prior to application of the facing. Another method of dealing with face stability problems includes placing of a flash coat of shotcrete. It is generally the case that face stability problems are likely to be most severe during the first one or two excavation stages, because of the presence of near-surface weathered and weakened materials or, in urban environments, the presence of loose fills or voids often associated with buried utilities.

Mass excavation is done with conventional earth moving equipment. Final trimming of the excavation face is typically done with a backhoe or hydraulic excavator. Usually, the exposed length of the cut is dictated by the area of face that can be stabilized and shotcreted in the course of a working shift. Ground disturbance during excavation should be minimized and loosened areas of the face removed before shotcrete facing support is applied. The excavated face profile should be reasonably smooth and regular in order to minimize subsequent shotcrete quantities.

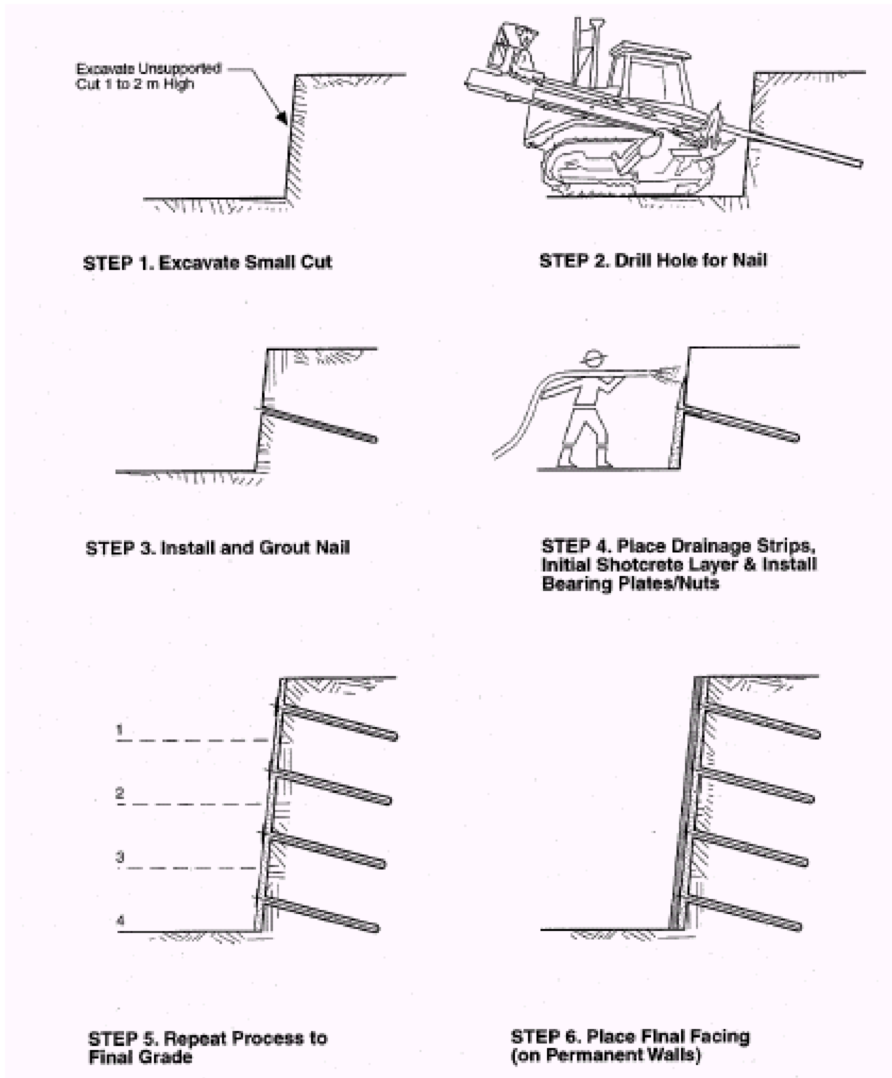


Figure 12.1 Soil Nail Wall Construction Sequence (FHWA, 1996)

A level working bench on the order of 10 m width is typically left in place to accommodate the drilling equipment used for nail installation. Smaller tracked drills are available that can work on bench widths as narrow as 5 m and with headroom clearance as low as 4 m. Larger bench widths may be necessary depending upon the equipment to be used during nail installation.

B. Drill Hole for Nail

Nail holes are drilled at predetermined locations to a. specified length and inclination using a drilling method appropriate for the ground. Drilling methods include both uncased methods for more competent materials (rotary or rotary percussive methods using air flush, and dry auger methods) and cased methods for less stable ground (single tube and duplex rotary methods with air or water flush, and hollow stem auger methods). Typical nail spacings are 1 to 2 m both vertically and horizontally. Typical nail lengths are 70 to 100 percent of the wall height and nail inclinations are generally on the order of 15 degrees below horizontal to facilitate grouting.

C. Install and Grout Nail

Plastic centralizers are commonly used to center the nail in the drillhole. However, where the nails are installed through a hollow stem auger, centralizers are generally ineffective and a stiffer (200 mm or lower slump) grout mix is used to maintain the position of the nail and prevent it from sinking to the bottom of the hole. The nails, which are commonly 20 to 35 mm bars (yield strength in range of 420 to 500 N/mm^3), are inserted into the hole and the drillhole is filled with cement grout to bond the nail bar to the surrounding soil. Grouting takes place under gravity or low pressure from the bottom of the hole upwards, either through a tremie pipe for open-hole installation methods or through the drill string (or hollow stem) or tremie pipe for cased installation methods.

For permanent nails, the steel bar is typically protected against corrosion damage with a heavy epoxy coating or by encapsulation in a grout-filled corrugated plastic sheathing.

D. Place Drainage System

A 400 mm-wide prefabricated synthetic drainage mat, placed in vertical strips between the nail heads on a horizontal spacing equal to that of the nails, is commonly installed against the excavation face before shotcreting occurs, to provide drainage behind the shotcrete face. The drainage strips arc extended down to the base of the wall with each excavation lift and connected either directly to a footing drain or to weep holes that penetrate the final wall facing. These drainage strips are intended to control seepage from perched water or from limited surface infiltration following construction. If water is encountered during construction, short horizontal drains are generally required to intercept the water before it reaches the face.

E. Place Construction Facing and Install Bearing Plates

The construction facing typically consists of a mesh-reinforced wet mix shotcrete layer on the order of 100 mm thick, although the thickness and reinforcing details will depend on the specific design. Following placement of the shotcrete, a steel bearing plate (typically 200 mm to 250 mm square and 15 mm thick) and securing nut are placed at each nail head and the nut is hand wrench tightened sufficiently to embed the plate a small distance into the still plastic shotcrete.

F. Repeat Process to Final Grade

The sequence of; excavate, install nail and drainage system, and place construction facing is repeated until the final wail grade is achieved. The shotcrete facing may be placed at each lift prior to nail hole drilling and nail installation, particularly in situations where face stability is a concern.

G. Place Final Facing

For architectural and long term structural durability reasons, a CIP concrete facing is the most common final facing being used for transportation applications of permanent soil nail walls. The CIP facing is typically structurally attached to the nail heads by the use of headed studs welded onto the beating plates. Under appropriate circumstances, the final facing may also consist of a second layer of structural shotcrete applied following completion of the final excavation. Pre-cast concrete panels may also be used as the final facing for soil nail walls.

Types of Nails (FHWA, 1996; ISSMFE-TC-17)

Conventionally, the steel reinforcing elements used for soil nailing can be classified as (a) driven nails and (b) grouted nails. However, specially designed corrosion-protected nails have also been used in permanent structures, specifically in aggressive environments. During the past decade the most significant technological innovations have been the development and use of the jet-grouted nails (Louis, 1986) and the launched soil nails (Ingold and Miles, 1996). A brief description of the available nailing systems is outline below:

Driven nails, commonly used in France and Germany, are small-diameter (15 to 46 mm) rods or bars, or metallic sections, made of mild steel with a yield strength of 350 MPa (50 ksi). They are closely spaced (2 to 4 bars per square meter) and create a rather homogeneous composite reinforced soil mass. The nails are driven into the ground at the designed inclination using a vibropercussion pneumatic or hydraulic hammer with no preliminary drilling. Special nails with an axial channel can be used to allow for grout sealing of the nail to the surrounding soil after its complete penetration. This installation technique is rapid and economical (4 to 6 per hour). However, it is limited by the length of the bars (maximum length about 20m) and by the heterogeneity of the ground (e.g., presence of boulders).

Grouted nails are generally steel bars (15 to 46 mm in diameter) with a yield strength of 60 ksi. They are placed in boreholes (10 to 15 cm in diameter) with a vertical and horizontal spacing varying typically from 1 to 3m depending on the type of the in-situ soil. The nails are usually cement-grouted by gravity or under low pressure. Ribbed bars can be used to improve the nail-grout adherence, and special perforated tubes have been developed to allow injection of the grout through the inclusion.

Corrosion-protected nails generally use double protection schemes similar to those commonly use in ground anchor practice. Proprietary nails have recently been developed by specialty French contractors (Intrafor-Cofor; Solrenfor) to be used in permanent structures. For permanent applications of soil nailing, based on current experience, it is recommended (Elias and Juran, 1991) that a minimum grout cover of 1.5 inches be achieved along the total length of the nail. Secondary protection should be provided by electrostatically applied resin-bonded epoxy on the bars with a minimum thickness of about 14 mils. In aggressive environments, full encapsulation is recommended. It may be achieved, as for anchors, by encapsulating the nail in corrugated plastic or steel tube grouted into the ground.

Jet-grouted nails are composite inclusions made of a grouted soil with a central steel rod, which can be as thick as 30 to 40 cm. A technique that combines the vibropercussion driving and high-pressure (greater than 20 MPa) jet grouting has been developed recently by Louis (1986). The nails are installed using a high frequency (up to 70 Hz) vibropercussion hammer, and cement grouting is performed during installation. The grout is injected through a small-diameter (few millimeters) longitudinal channel in the reinforcing rod under a pressure that is sufficiently high to cause hydraulic fracturing of the surrounding ground. However, nailing with a significant lower grouting pressure (about 4 MPa) has been used successfully, particularly in granular soils. The inner nail is protected against corrosion using a steel tube. The jet-grouting installation technique provides recompaction and improvement of the surrounding ground and increases significantly the pull-out resistance of the composite inclusion.

Launched Nails - The nail launching technology (Bridle and Myles, 1991; Ingold and Myles, 1996) consists of firing directly into the ground, using a compressed air launcher, nails of 25mm and 38mm in diameter, made from bright bar (EN3B to BS982) with nail lengths of 6 meters or more. The nails are installed at speeds of 200 mph with an energy transfer of up to 100kJ. This installation technique enables an optimization of nail installation with a minimum of site disruption. During penetration the ground around the nail is displaced and compressed. The annulus of compression developed reduces the surface friction and minimizes damage to protective coatings such as galvanized and epoxy. The technology is presently used primarily for slope stabilization although successful applications have also been recorded for retrofitting of retaining systems. However, a rigorous evaluation of the pull-out resistance of launched nails is required prior to their use in retaining structures..

Grouting (Goldstein, 2001; FHWA, 1996)

Neat cement grout with a water-to-cement ratio of about 0.4 to 0.5 is usually used. In many cases for open-hole drilling, the low-pressure tremie method works well. In Germany, the nail may be installed with a regrout pipe attached, and the grout is added under pressure, fracturing the initial grout and creating a better bond between the grout and the soil. In general, grout may be added either before or after installation of the nail.

Facing (Goldstein, 2001)

Once the nails are installed and grouted, a shotcrete facing between 3 and 6 in. thick is applied, with a welded wire mesh at mid-thickness. This is generally used for temporary wall facings. Permanent walls may receive a shotcrete cover of up to 10 in. thick, usually with a second layer of wire mesh. In both of these cases, the facing is not considered to be a structurally significant supporting part of the wall. Shotcrete suitable for facings has been produced by either dry or wet mix process. Both dry mix and pneumatic feed wet mix use a stiff mixture (water: cement ratio = 0.4).

The experience in France indicates that nail loads at the facing generally do not exceed 30-40% of the maximum loads in the nail, so they recommend a facing designed for a uniform wall pressure equal to 60% of the maximum nail load on a nail spacing of 3 ft. For walls with greater nail spacing (e.g., 10 ft.), the facing should be designed for 100% of the maximum nail load.

Permanent structures can be made more pleasing to the eye with the addition of cast-in-place concrete facings with a minimum of 8-in. thickness. Precast decorative panels may also be attached directly to the shotcrete facing.

Advantages of Soil Nailing (FHWA, 1996)

Soil nailing exhibits many of the same advantages as tieback walls as a method of ground support/reinforcement, together with additional benefits that are unique to nailing. Like tieback walls, the top down construction technique of soil nailing offers the following benefits:

- Improved economy and lessened environmental impact compared to conventional retaining walls, through the elimination of the need for a cut excavation and backfilling.

  No backfill is required for the soil nailed wall, whereas a considerable amount is needed for conventional or MSE retaining walls.

- Improved economy and materials savings through the incorporation of the temporary excavation support system into the permanent support system.
- Improved economy and lessened environmental impact through reduction in the right-of-way (ROW) requirements.
- Improved safety by eliminating cramped excavations cluttered with internal bracing.

  Compared to tieback walls, soil nailing may offer the following advantages in ground suitable for soil nailing:

- Elimination of the need for a high capacity structural facing (i.e., soldier piles and thick CIP) since the maximum earth pressure support loads are not transferred to the excavation face. For constructability reasons, a permanent CIP facing is normally 200 mm thick. Most tieback walls have a permanent facing that is 250-300 mm thick.
- Improved construction flexibility in heterogeneous soils with cobbles, boulders or other hard inclusions, as these obstructions offer fewer problems for the relatively small diameter nail drill-holes than they do for the. large diameter soldier pile installations.
- Improved construction flexibility where overhead access is limited (e.g., road widening under an existing bridge) through the elimination of the requirement for drilled or driven soldier piles installed through the bridge deck or in hand dug pits.
- Ease of construction and reduced construction time - soldier pile installations are not required, soil nails are not prestressed, and construction equipment is relatively small, mobile, and quiet. This is particularly advantageous on urban sites.
- The vertical components of the nail reaction at the facing are smaller than those for tiebacks and are also distributed more evenly over the entire excavation face. This eliminates the need for significant wall embedment below grade, such as is required for tieback soldier piles.
- Higher system redundancy as the soil nails are installed at a far higher density than the prestressed tieback anchors, and the consequences of a unit failure are therefore correspondingly less severe. It should be noted that this does not necessarily imply higher

system reliability for soil nail walls, since each tieback is tested during installation, whereas only a small percentage of nails arc tested.

- Reduced right-of-way requirements, as the nails are typically shorter than the tieback anchors.

Other favorable features of soil nail retaining systems include the following:

- The method is well-suited to sites with difficult or remote access because of the relatively small size and the mobility of the required construction equipment.
- The method is well-suited to urban construction where noise, vibration, and access can pose problems.
- The construction method is flexible and can follow difficult excavation shapes using splayed nails and can cope with significant variations in soil conditions encountered during construction. Nail layout modifications during construction (e.g., moving nails to miss unanticipated obstructions) can be relatively easily accomplished.
- The system is relatively robust and flexible and can accommodate significant total and differential settlements. Soil nail retaining walls have been documented to perform well under seismic loading conditions [Felio,1990].
- Field monitoring has indicated that overall movements required to mobilize the reinforcement forces are relatively small and correspond generally to the movements that would be expected for well braced systems (Category I) in Peck's classification [Peck, 1969].

  Measured wall movements are usually in the range of 0.1 to 0.3 percent of the wall height.
- The method is well suited to specialist applications such as the rehabilitation of distressed retaining structures.

Limitations of Soil Nailing (FHWA, 1996)

Soil nailing and other cut retaining techniques share the following limitations:

- Permanent underground easements may be required.
- In urban areas, the closely spaced array of reinforcements may interfere with nearby utilities. Utility trenches represent potential planes of weakness that can contribute to failure, may contain poorly compacted or otherwise unsuitable fill for soil nailing, and may also carry ground water to the wall. Significant groundwater seepage at the excavation face can cause serious constructability problems.
- Horizontal displacements may be somewhat greater than with prestressed tiebacks, and this may cause distortions to immediately adjoining structures.
- Nail capacity may not be economically developed in cohesive soils subject to creep, even at relatively low load levels.
- The long-term performance of shotcrete facings has not been filly demonstrated particularly in areas subject to freeze-thaw cycles.

The technique also has certain practical limitations to its application. These are:

- Soil nail construction requires the formation of cuts generally 1 to 2 m high in the soil.

  These must then stand unsupported, prior to shotcreting and nailing. The soil must therefore have some natural degree of "cohesion" or cementing, otherwise slotting, berming or reduced cut excavation lift heights may be necessary to stabilize the face, adding both complication and cost. Therefore, soil nailing is not well suited to applications in clean sands and gravels.

- A dewatered face in the excavation is highly desirable for soil nailing. If the ground water percolates through the face, the unreinforced soil may slump locally upon excavation, or the shotcrete to soil bond may be reduced, making it impossible to establish a satisfactory shotcrete skin.

- Excavations in soft clay arc also unsuited to stabilization by soil nailing. The low frictional resistance of soft clay would require a very high density of in-situ reinforcement of considerable length to ensure adequate levels of stability. Tieback or bored pile walls are more suited to these conditions.

- Soil nailing in sensitive or expansive clays must be carefully evaluated. Care must be taken to prevent disturbing the soil or allowing water to soften and weaken the soil.

Finally, wall performance can be relatively sensitive to the selected method of construction, and is best achieved by experienced, specialty contractors.

Application Criteria (FHWA, 1996)

The most cost-effective application of soil nail retaining walls is usually as an alternative to tieback soldier pile walls or conventional retaining walls with temporary shoring i.e., where site geometry or adjacent property constraints do not permit an unsupported permanent cut excavation. Soil nail walls are particularly well-suited to the following highway applications, all of which have been successfully demonstrated on highway projects in both North America and Europe roadway cut excavations, widening under an existing bridge end, stabilization and repair and reconstruction of existing retaining structures. Examples of soil nailing applicability are:

Retaining Structures in Cuts

In ground suitable for soil nailing, soil nailing technology can be considered for permanent or temporary cut wall applications where conventional cast-in-place, tieback wall, precast, or mechanically stabilized earth (MSE) structures are applicable. Soil nailing is considered to be particularly applicable for uphill widening projects that must be constructed either within an existing ROW or in steep terrain.

End Slope Removal Under Existing Bridge Abutment

For underpass widening through removal of the bridge abutment end slope, soil nailing offers the major advantage of not requiring soldier pile installation. Because of limited headroom conditions beneath the bridge structure, soldier piles must generally be installed through the existing bridge deck, with significant disruption to the overpass traffic and increased cost. In this application, soil nailing provides both the temporary and permanent earth support function. If lateral displacements are of particular concern (e.g., adjacent a bridge spread footing), the upper nail rows immediately adjacent the footing may be installed in slots to help limit and control the displacements.

Repairs and Reconstruction of Existing Retaining Structures

Soil nails can be installed through existing retaining walls and the technique is finding application in the stabilization or strengthening of existing failing or distressed retaining structures. This type of application represents something of a departure from the original soil nailing concept of excavate and support, in that the ground deformations required to mobilize the reinforcing loads do not derive from removal of lateral support during excavation but from on-going movements associated with the distressed structure. In this context, soil nailing can also be similarly used to stabilize marginally stable slopes.

Feasibility Evaluation (FHWA, 1998)

Geometric Constraints

The required length of the nails can vary between 60 and 120 percent of the wall height; however, more typically 70 to 90 percent. Consequently, sufficient right-of-way must be available for the nails. In urban areas, underground utilities may interfere with the upper rows of nails, which are typically installed 3 to 5 ft below the wall top.

Soil and Groundwater Conditions

Soil nail walls are constructed in ground where a 3- to 6.5-ft vertical slope can stand without support for several days during construction and is stable during the few hours it takes to drill and insert the nails. The depth of the cut layer depends on the soil's ability to stand unsupported while the nails are being inserted.

1. Weathered rock, talus slope deposits, silts, clays with low plasticity that are not prone to creep, naturally cemented sands and gravels, heterogeneous and stratified soils, and some kinds of fine-to-medium homogeneous sand are suitable for soil nail construction.
2. Soils not conducive to soil nail technology are soft plastic clays; peat/organic soils; loose, low-density, and/or saturated soils; and coarse sand and gravels that are uncemented or lack capillary cohesion.
3. Organic soils, cinder, slag or ash fills, and acid mine wastes are not suitable
4. Cohesive soils with LL > 50 and PI > 20 are not suitable
5. SPT – N values should be greater than 5 blows
6. Unconfined compressive strengths should be greater than 50 kPa (1 ksf)
7. In uniform cohesionless soils ($C_u < 2$), are susceptible to sloughing of the initial cut. Consequently, cut face stabilization by grouting or shotcreting during or before excavation is required. A temporary face stabilizing berm is another alternative.

Drainage must be controlled, and construction below the permanent ground water table is impossible unless a complete drainage system is installed. Most commonly, face drainage is used consisting of a 400-mm-wide prefabricated geotextile drainage strip is placed behind the shot-crete wall covering the nailed structure. The drainage strips are installed from the top down as construction proceeds on a horizontal spacing equal to the nail spacing and discharging into a base drain or weep hole. Weep holes are typically 50-to-100mm diameter PVC pipes. The water is collected at the wall base and channeled away using perforated pipe. Alternatively, weep holes can be made through the face of the wall, used with or without perforated drainpipes.

Aggressive Soils and Corrosion

Corrosion prevention is necessary in permanent structures and in "aggressive" soils, which are defined as having a pH below 4.5, a resistivity below 2,000 ohm-cm, sulfate levels above 200 ppm, and chloride levels above 100 ppm. If these conditions exist, corrosion-protected nails must be used.

The German approach to corrosion protection is considered conservative and is preferred. It involves using nails with "double corrosion protection," in which the steel nail is encapsulated in a corrugated plastic sheath (> 40 mil) and cement grout annulus. The double coating prevents damage even if small cracks occur in the cement grout. This double corrosion protection is required for permanent structures and for temporary structures in aggressive ground intended to last more than 30 years. Epoxy coatings or grouts are not recommended and are far more expensive than the double corrosion system described above. Further, research indicates that under no circumstances should stainless steel reinforcing strips be used in aggressive ground. In France, a structure less than 10 years old failed using this method of reinforcement.

Deformations (FHWA, 1996)

During construction of a soil nail wall from the top-down, the reinforced soil zone tends to rotate outwards about the toe of the wall as part of the process of mobilizing tensile loads within the nails. Hence, maximum horizontal movements occur at the top of the wall and decrease progressively towards the toe of the wall. Settlements at the facing also occur, and these tend to be on the same order of magnitude as the horizontal movements at the top of the wall as shown in Table 12.1. Displacements of the facing depend on the following factors (French National Research Project CLOUTERRE 1991)

- Construction rate;
- Nail spacing and excavation lift heights;
- Nail and soil stiffness;
- Global factor of safety;
- Nail inclination (greater displacements for greater inclinations because of less efficient reinforcing action);
- Bearing capacity of the foundation soils; and
- Magnitude of any surcharge loadings.

Table 12.1 Estimated Structure Deflection (FHWA, 1998)

	Weathered Rock	**Sand**	**Clay**
Vertical of Horizontal Deflection at top of wall face	H/1000	2H/1000	4H/1000

COSTS (FHWA, 1998)

For temporary construction, an estimate of \$170/m^2 to \$400/m^2 can be assumed. For permanent construction with a pre-cast of cast-in-place concrete facing, estimate \$400/m^2 to \$600/m^2.

Engineering Behavior and Design Concepts

The basic design concept of soil-nailed retaining structures relies upon the transfer of resisting tensile forces generated in the inclusions into the ground through friction at the interfaces. The frictional interaction between the ground and the quasi "nonextensible" steel inclusions restrain the ground movement during and after excavation. The resisting tensile forces mobilized in the inclusions induce an apparent increase of normal stresses along potential sliding surfaces increasing the overall shear resistance of the native ground. The main engineering concern in the design of these retaining systems is to ensure that ground-inclusion interaction is effectively mobilized to restrain ground displacements and can secure the structure stability with appropriate factor of safety.

Design Methods for Nailed Soil-Retaining Structures (ISSMFE-TC-17)

The design procedure for a soil-nailed retaining structure should include the following steps:

1. For the specified structure geometry (depth and cut slope inclination), ground profile, and boundary (surcharge) loadings, estimate working nail forces and location of the potential sliding surface.
2. Select the reinforcement type (type, cross-sectional area, length, inclination, and spacing) and verify local stability at each reinforcement level, that is, verify that nail resistance (strength and pull-out capacity) is sufficient to withstand the estimated working forces with an acceptable factor of safety.
3. Verify that the global stability of the nailed-soil structure and the surrounding ground is maintained during and after excavation with an acceptable factor of safety.
4. Estimate the system of forces acting on the facing (i.e., lateral earth pressure and nail forces at the connection) and design the facing for specified architectural and durability criteria.
5. For permanent structures, select corrosion protection relevant to site conditions.
6. Select the drainage system for groundwater piezometric levels.

The available design methods for soil nailed retaining structures, can be broadly classified into two main categories.

1. Limit equilibrium design methods or modified slope stability analyses, which are used to evaluate the global safety factor of the nailed structures with respect to a rotational or translational failure along potential sliding surfaces, taking into account the shearing, tension, or pull-out resistance of the inclusions crossing the potential failure surface.
2. Working stress design methods, which are used to estimate the tension and shear forces generated in the nails during construction under the design loading conditions, and evaluate the local stability at each level of nails.

A detailed discussion of the available design methods was provided by Elias and Juran, (1991); Juran and Elias, (1991); CLOUTERRE, (1991); Xanthakos, et al. (1994); FHWA, (1996). A particular emphasis has been placed by different investigators (Mitchell, et al. 1987; Elias and Juran, 1991; Gassler, 1993; CLOUTERRE, 1991; Schlosser, et al. 1993; Plumelle, 1993; Thompson and Miller, 1990) on the evaluation of the available design methods through comparisons of method predictions with full scale experiments and measurements on in-service structures.

In soil nailing, similarly to ground anchors, the load transfer mechanism and the ultimate pull-out resistance of the nails depend primarily upon soil type and strength characteristics, installation technique, drilling method, size and shape of the drilled hole, as well as grouting method and pressure used.

To date, estimates of the pull-out resistance of nails are mainly based upon empirical formulae (or ultimate interface shear stress values) derived from field experience. These formulae are useful for feasibility evaluation and preliminary design. Table 12.2 (Elias and Juran, 1991) provides a summary of estimated ultimate interface shear stress values for soil nails as a function of soil type and installation technique.

FHWA SOIL NAILING DESIGN CHARTS (FHWA, 1996)

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 12.2.

Table 12.2 Estimated Ultimate Interface Lateral Shear Stress Values for Soil Nails

Grouted Construction Method	Soil Type	**Soil Nailing** (Elias and Juran, 1991) **Ultimate Lateral Shear Stress, kip/ft**
Rotary drilled	Silty sand	2 to 4
	Silt	1.2 to 1.6
	Piedmont residual	1.5 to 2.5
Driven casing	Sand	6
	Dense sand/gravel	8
	Dense moraine	8 to 12
	Sandy colluvium	2 to 4
	Clayey colluvium	1 to 2
Jet grouted	Fine sand (medium dense)	
	Sand	8
	Sand/gravel	20
Augured	Soft clay	0.4 to 0.6
	Stiff to hard clay	0.8 to 1.2
	Clayey silt	1 to 2
	Calcareous sandy clay	4 to 6
	Silty sand fill	0.4 to 0.6

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.

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 12.3.

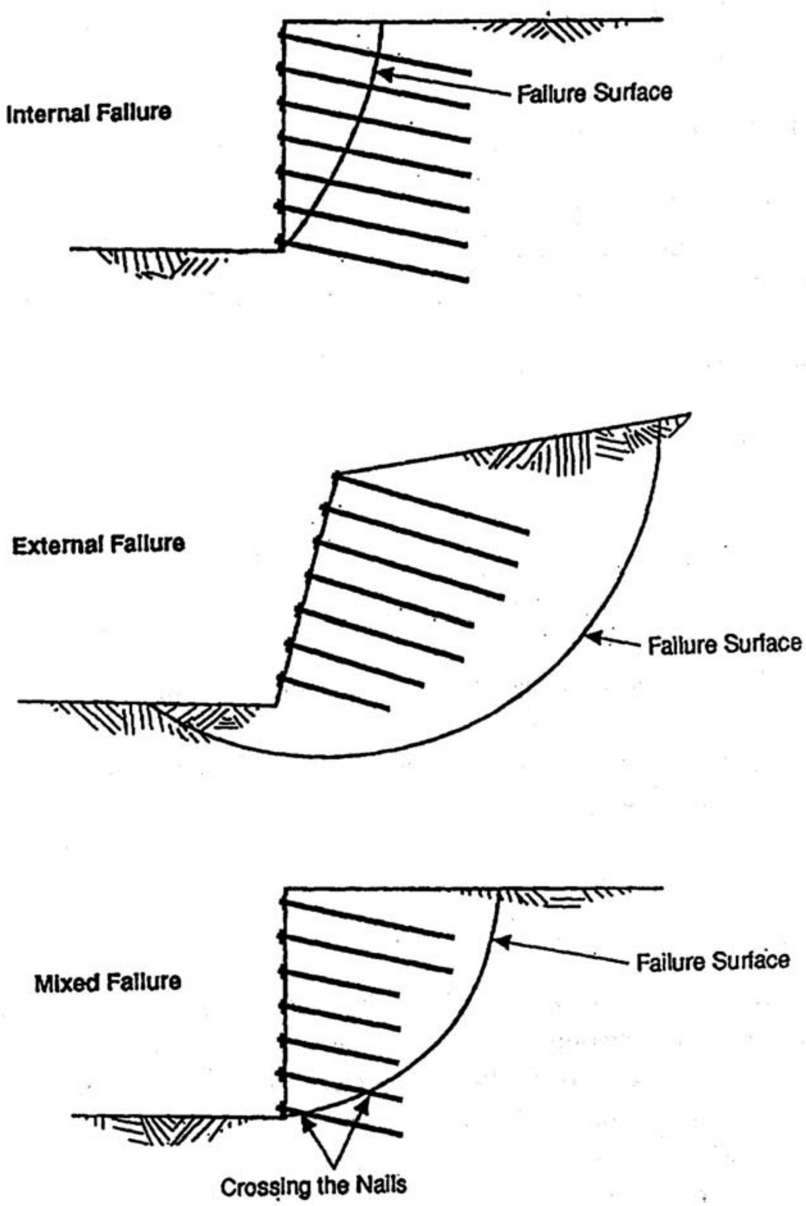


Figure 12.2 Different Types of Failure Surfaces to be Analyzed for Soil Nailed Walls
(FHWA, 1998)

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.

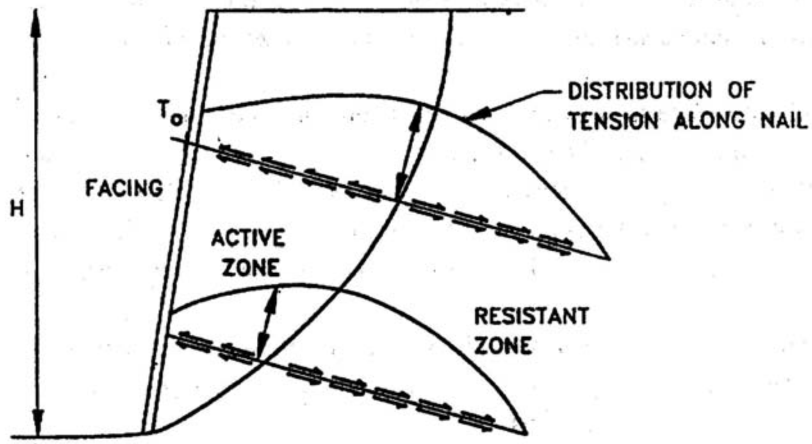


Figure 12.3 Conceptual Soil Nail Behavior (FHWA, 1998)

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.

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, 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 12.3 through 5.

Table 12.3 Estimated Pullout Strengths in Rock

Construction Method	Rock Type	Ultimate Pullout Strength F_t (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 12.4 Estimated Pullout Strength in Cohesionless Soils

Construction Method	Soil Type	Ultimate Pullout Strength F_t (KPa)
Rotary Drilled	Silty sand	100 – 150
	Silt	60 – 75
	Piedmont residual	40 – 120
	Fine colluvium	75 – 150
	Sand/gravel	100 – 180
Driven Casing	Sand/gravel (low overb.)	190 – 240
	Sand/gravel (high overb.)	280 – 430
	Dense Moraine	380 – 480
	Colluvium	100 – 180
Jet Grouted	Sand	380
	Sand/gravel	700
Augered	Silty sand fill	20 – 40
	Silty fine sand	55 – 90
	Silty clayey sand	60 – 140

Table 12.5 Estimated Pullout Strength in Cohesive Soils

Construction Method	Soil Type	Ultimate Pullout Strength F_t (KPa)
Augered	Loess	25 – 75
	Soft clay	20 – 30
	Stiff clay	40 – 60
	Clayey silt	40 – 100
	Calcareous sandy clay	90 – 140
Driven casing	Clayey silt	90 – 140
Rotary drilled	Sitly clay	35 – 50

DESIGN OF STRUCTURES

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. 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.

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 12.4 through 12.12 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 0.20 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.

- Dimensionless Cohesion, C_D

  C_D is the soil cohesion normalized with respect to the soil unit weight and the vertical height of the cut.

  $$C_D = c/(Fc\gamma H)$$

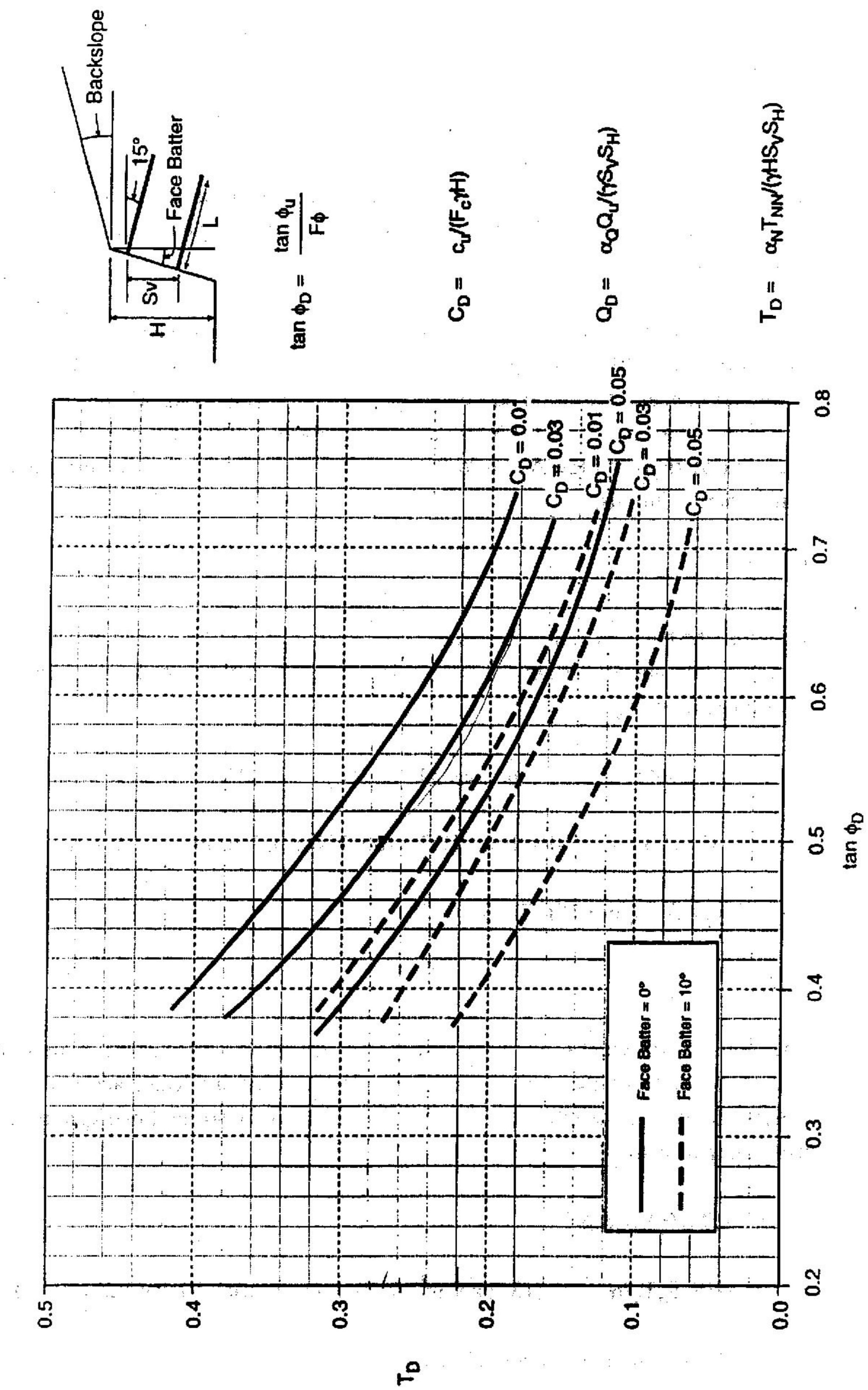


Figure 12.4 Design Chart Set 1: Backslope = 0°

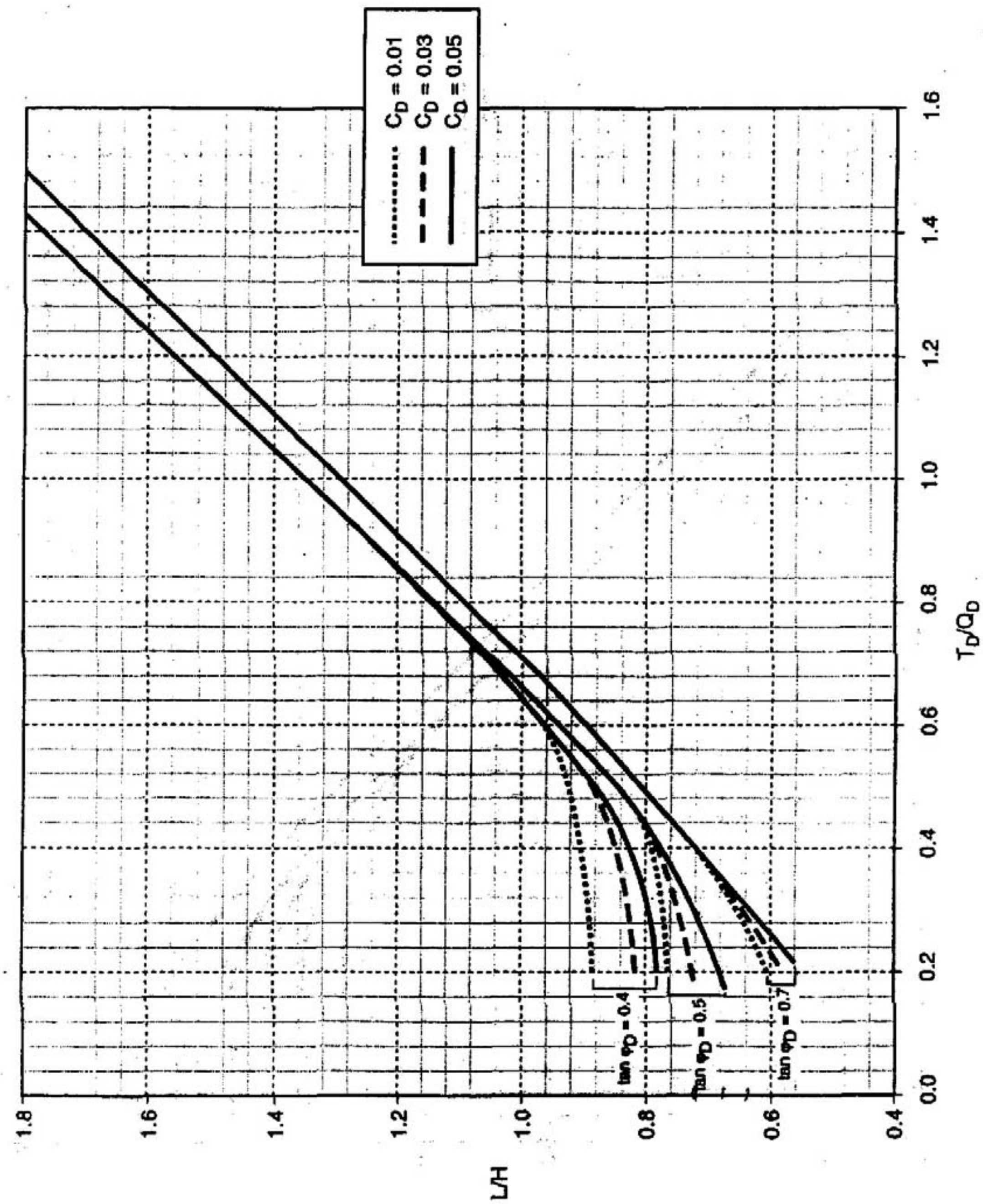


Figure 12.5 Design Chart Set 1: Backslope = 0°, Face Batter = 0°

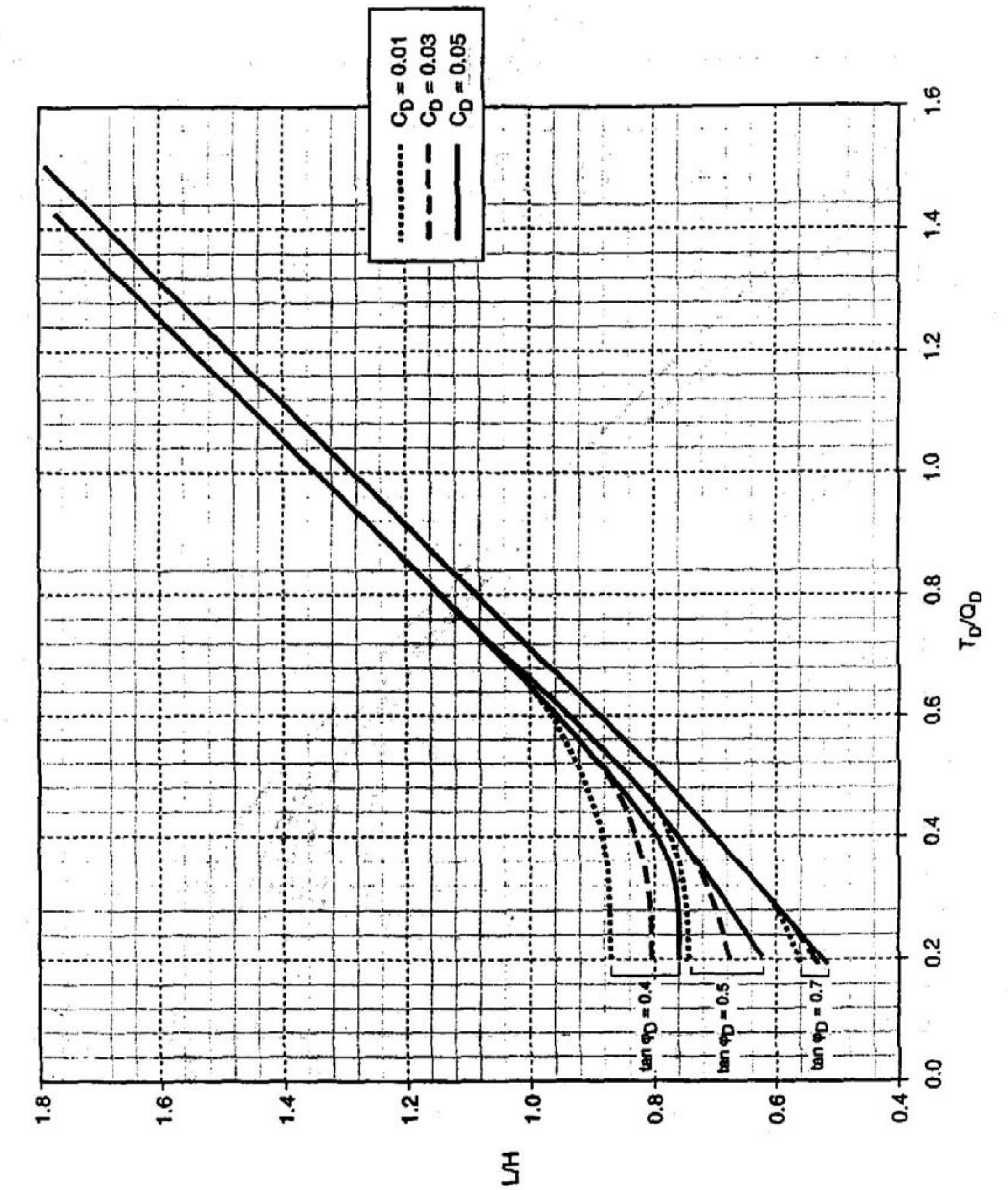


Figure 12.6 Design Chart Set 1: Backslope = 0°, Face Batter = 10°

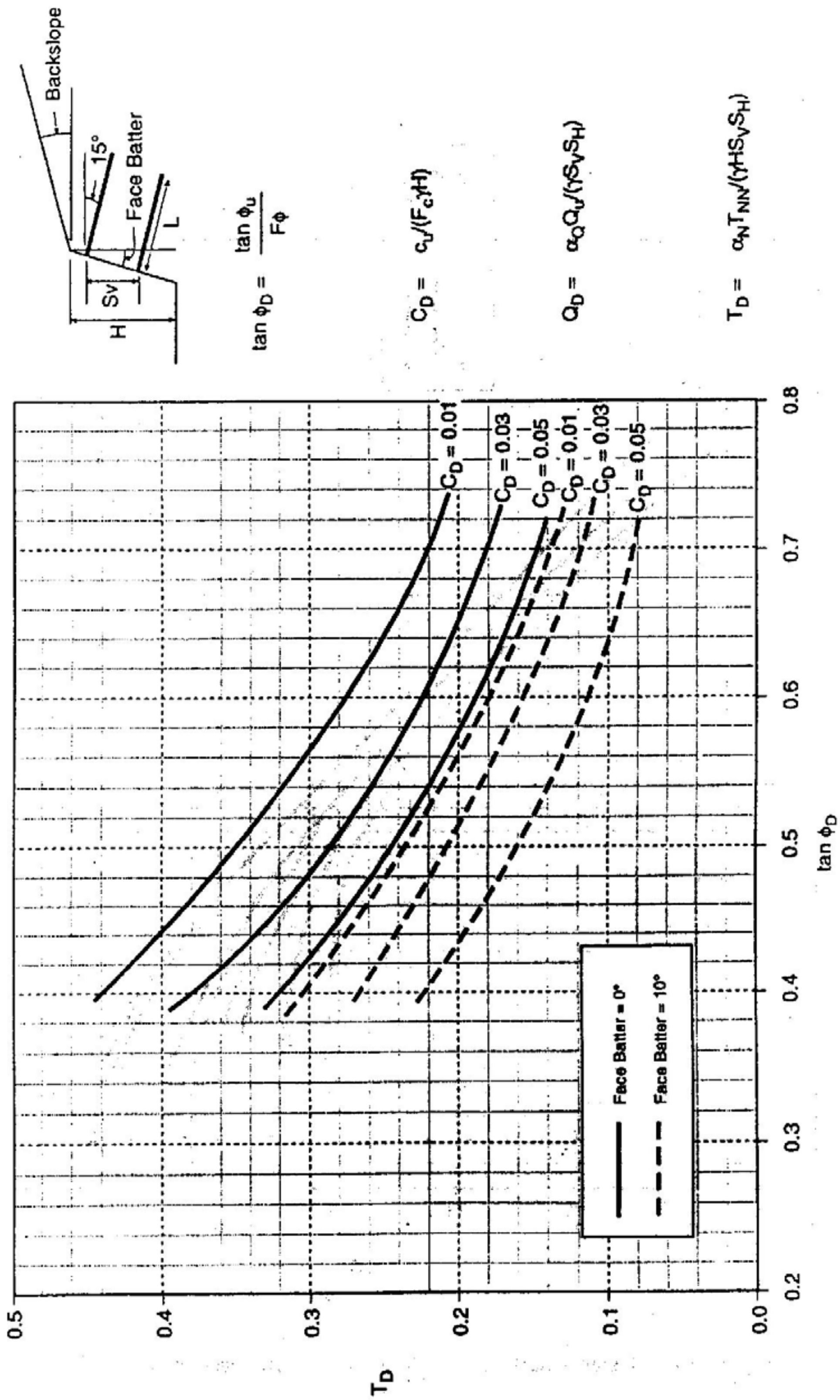


Figure 12.7 Design Chart Set 2: Backslope = 10°

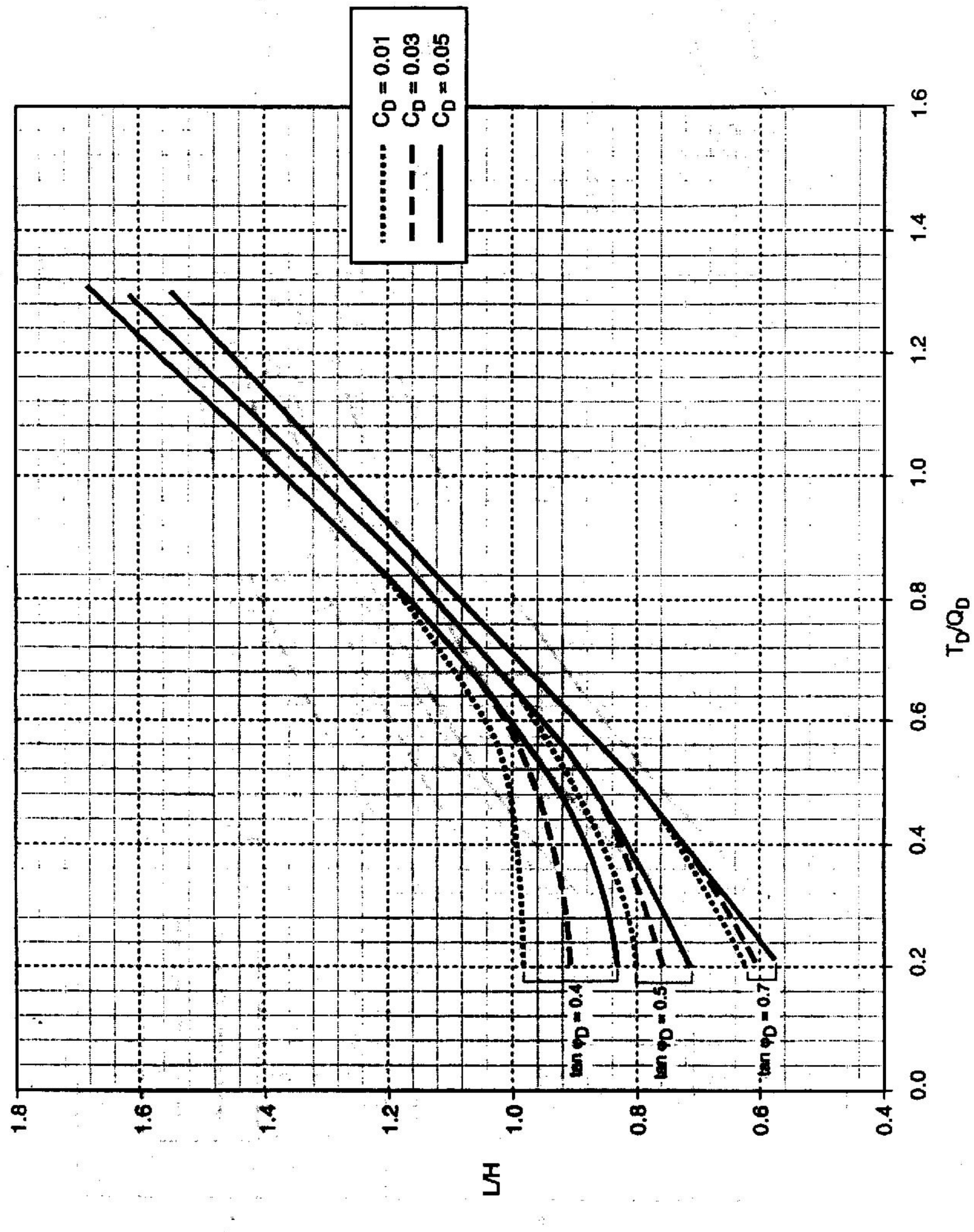


Figure 12.8 Design Chart Set 2: Backslope = 10°, Face Batter = 0°

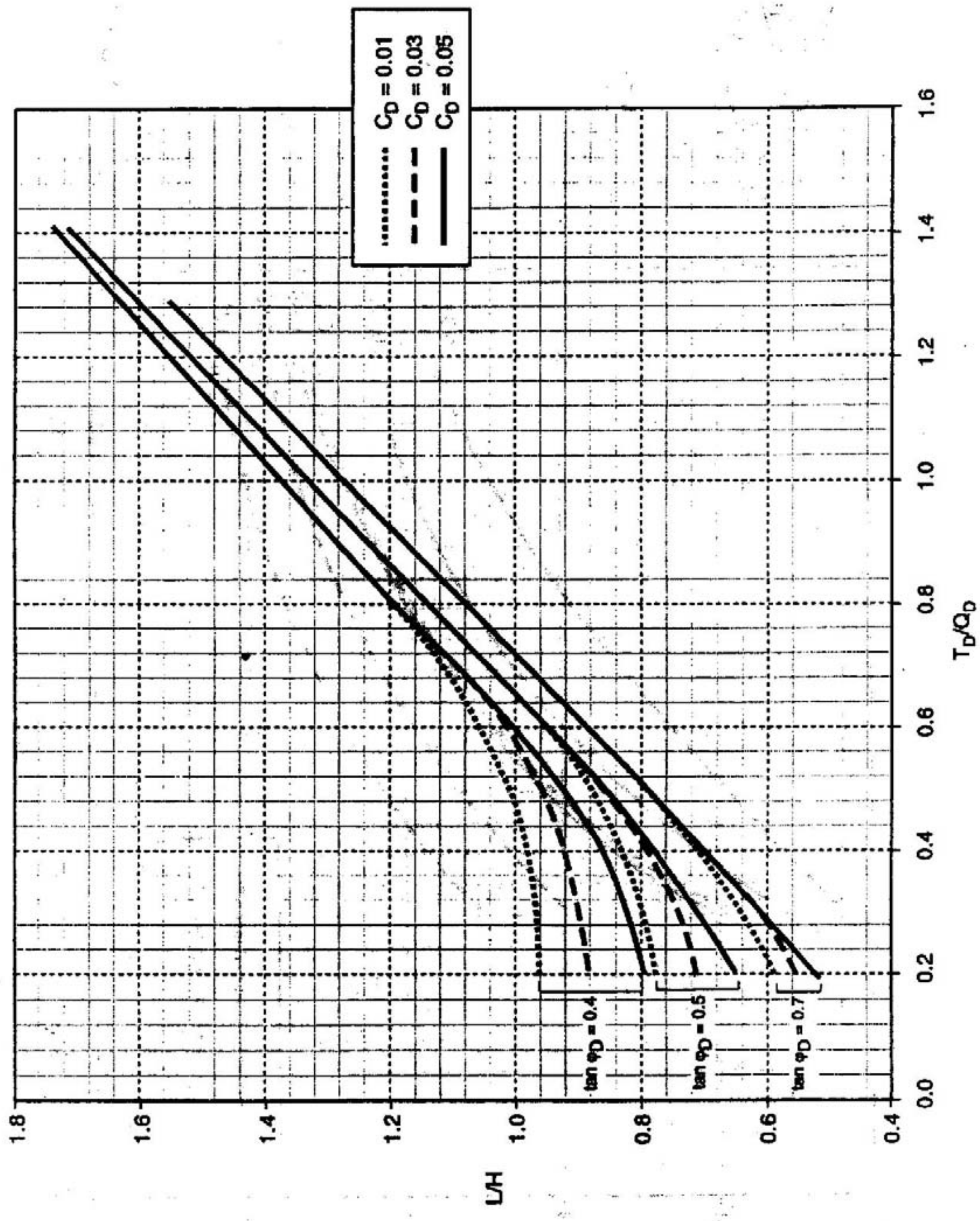


Figure 12.9 Design Chart Set 2: Backslope = 10°, Face Batter = 10°

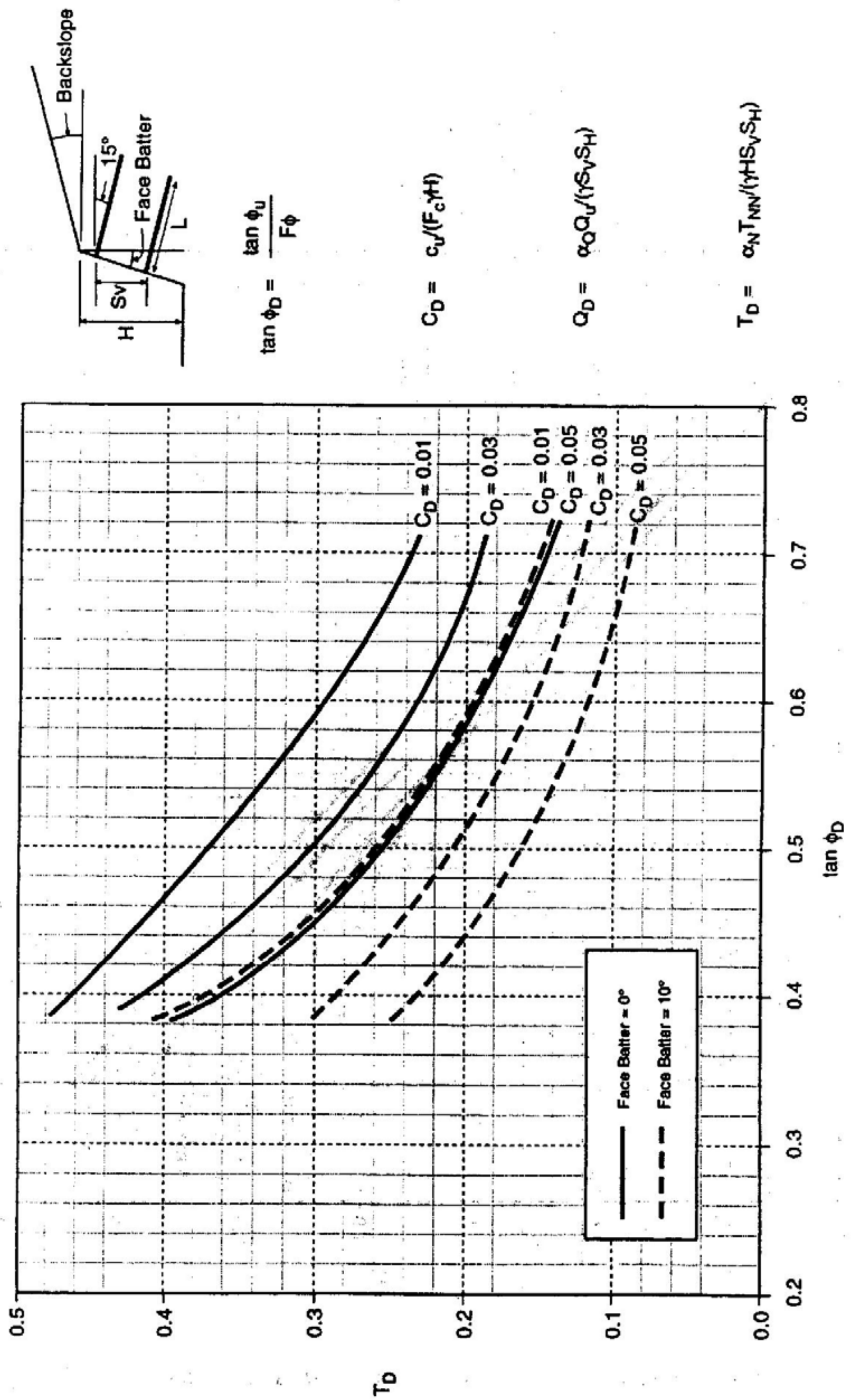


Figure 12.10 Design Chart Set 3: Backslope = 20°

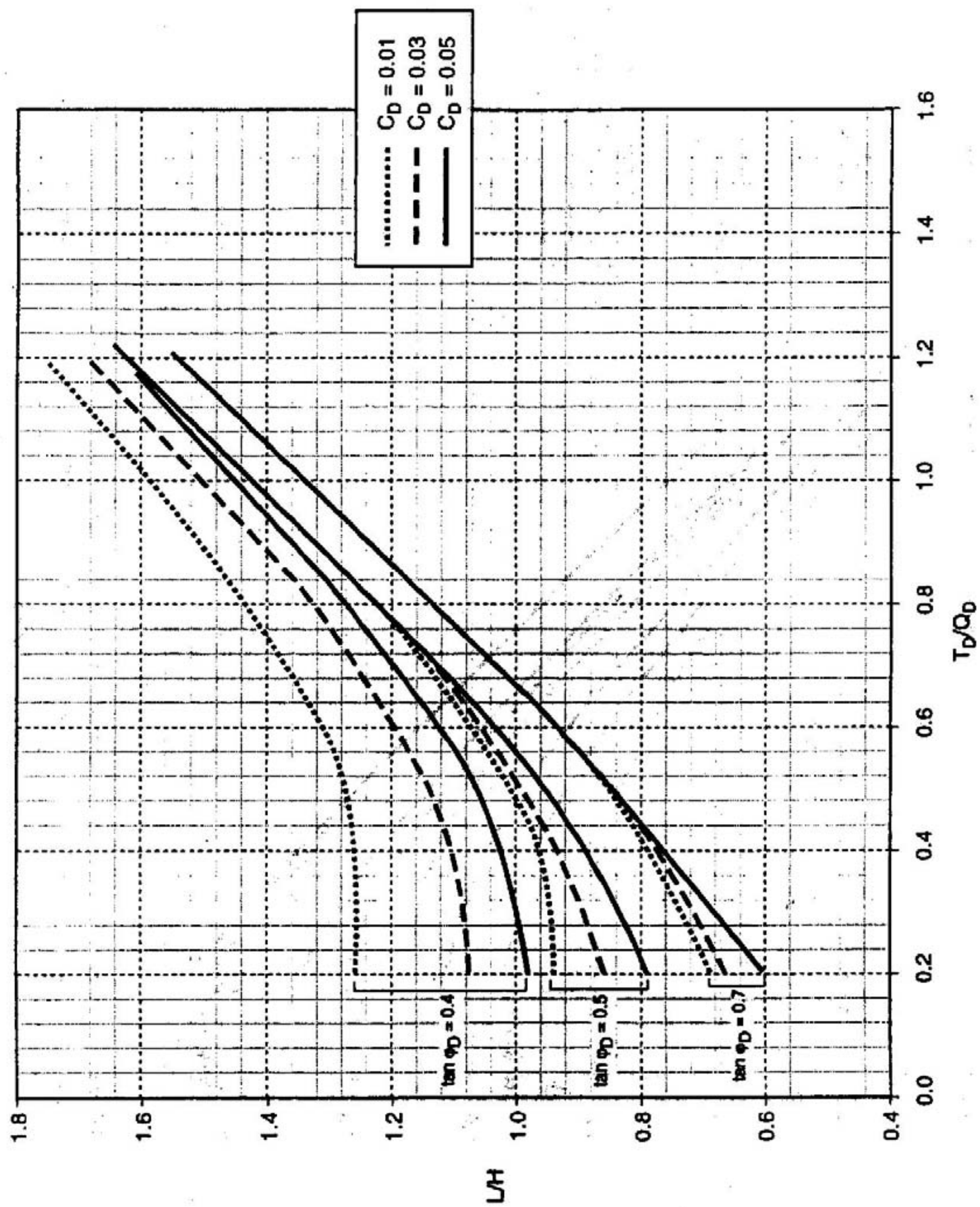


Figure 12.11 Design Chart 3: Backslope = 20°, Face Batter = 0°

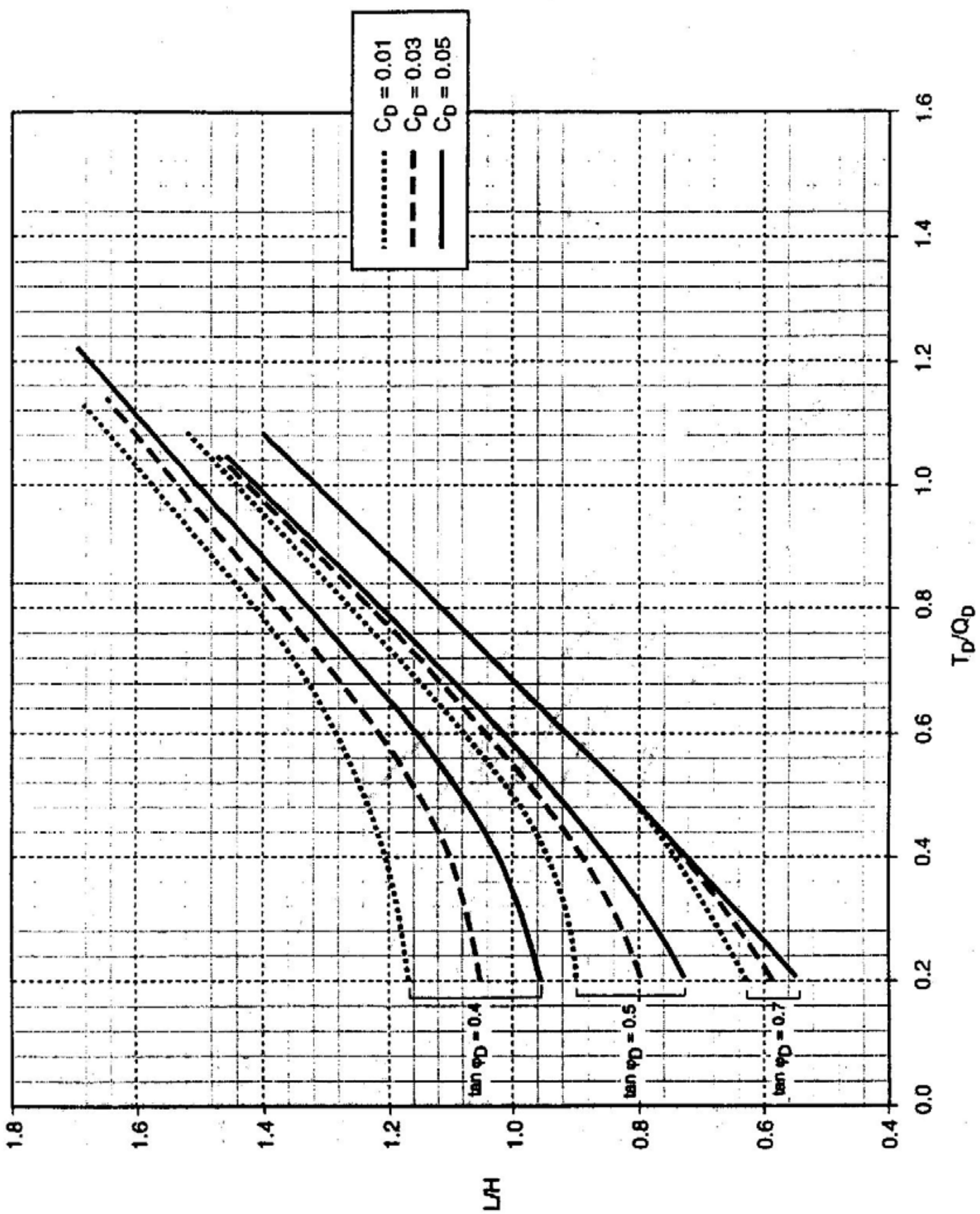


Figure 12.12 Design Chart 3: Backslope = 20°, Face Batter = 10°

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 (UN) nail yield strength normalized with respect to the soil specific weight (γ), the vertical height of the slope (H), and the nail spacings (SV,SH):

  $TD = \alpha T_N/(\gamma H S_V S_H)$ $\alpha_N = 0.55$

  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 spacing

  $Q_D = \alpha_Q Q_u/\gamma S_v S_H)$ $\alpha_D = 0.50$

  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,

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 Qu = 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.

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.

$$\phi_D = \tan^{-1}[\tan(\phi)/F_\phi = \tan^{-1}[\tan(34°)/1.35] = 26.5°$$

$$c_D = c/(Fc\gamma H) = (5.0 \text{ kN/m}^2)/[1.35(18.0 \text{ kN/m}^3)(9.50\text{m})] = 0.022$$

From Chart A (Figure 12.4), T_D = 0.23

Step 2

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.

$$T_D = \alpha_N T_N /(\gamma H S_V S_H)$$

$$T_N = \gamma H S_v S_H T_D / \alpha_N = (18.0\ kN/m^3)(9.50\ m)(1.50\ m)(1.50\ m)(0.23)/(0.55)$$

TN = 161 kN (Required nominal nail strength)

Step 3

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.

$$Q_B = \alpha_Q Q_U/(\gamma S_V S_H) = (0.50)(60.0\ kN/m)/[(18\ kN/m^3)(1.50\ m)(1.50\ in)] = 0.74$$

$$T_D / Q_D = 0.23/0.74 = 0.31$$

From Chart B (Figure 12.5), L/H = 0.87

$$L = 0.87\ (9.50m) = 8.3m$$

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.

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	AASFITO T-288
Sulfate	Above 200 ppm	AASHTO T-290
Chloride	Above 100 ppm	AASI-ITO 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.

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_0), as a function of nail spacing. The nail tensile force at the face has been approximated empirically from French research as:

$$To/T_{max} = 0.5 + (S-0.5)/S \quad \text{for } 1\text{ m} \le S \le 3\text{ m}$$

$$To/T_{max} = 0.6 \quad \text{for } S \le 1\text{ m}$$

$$To/T_{max} = 1.0 \quad \text{for } S \ge 3\text{m}$$

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 Pacing. 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 CW wall cap 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.

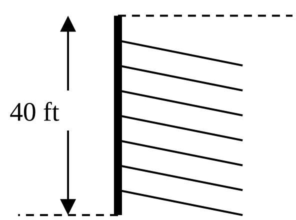
Separate Fascia Wail (CIP or Precast Panels). As an alternative to the exposed shotcrete finish, the shotcrete can be covered with a separate fascia wall consisting either of a CW wall or precast, face panels. The CIP section is typically a minimum 200 mm thick for constructability and shear stud connectors are welded to the nail cover plates to transfer load.

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.

SOIL NAIL EXAMPLE

The following example is an illustration using the FHWA (1998) soil nail design charts. (Figs. 12.4 through 12.12). The design is to determine: (a) nail length, (b) nail strength (diameter), and (c) facing thickness for a 40ft high wall. Nails are at 15° inclination.

Design FS = 1.2; i.e., $F_\phi = F_c = 1.2$



Soil Properties
γ = 120pcf
ϕ = 36° , c = 180 psf
f_l = 3000 psf

Nail Properties
$S_h = S_v$ = 5 ft
D_{grout} = 5 inches
D_{nail} = 1 inch , f_y =60ksi

1. Determine factored friction angle, $\phi_{design} = \text{Tan}^{-1}(\tan\phi / FS) = \tan^{-1}(36° / 1.2) = 31°$
2. Determine factored cohesion, $C_{design} = C_u / (F_c\, \gamma H)$ = 180 psf / (1.2 x 120 x 40) = 0.03
3. Using Figure 12.4 with a face batter = 0° and backslope = 0°, using $\phi_d = 31° \rightarrow \tan(31°) = 0.6$ and $C_d = 0.03$ gives $T_d = 0.21$
4. T_d = dimensionless nail tension capacity: $T_d = \dfrac{\alpha_n T_N}{\gamma H S_H S_v}; \alpha = 0.55$

   $\therefore$
   $$T_N = \frac{(0.21)(120 \times 40 \times 5 \times 5}{0.55} = 45,818\text{lbs}$$

   Considering f_y = 60ksi; $A_{steel} = 45.8^K$ / 60ksi = 0.76 in^2 $\therefore D = \sqrt{4A_s/\pi} \approx 1.0''$ (# 8 bar)

5. Find dimensionless pullout resistance: $Q_d = \dfrac{\alpha_Q Q_u}{\gamma S_H S_V}; \alpha_Q = 0.5$

$$Q_d = \frac{(0.5)(3000psf)}{120 \times 5 \times 5} = 0.5$$

6. Using Figure 12.11: T_d / Q_d = 0.21 / 0.5 = 0.42; and Tan ϕ_d =0.6→ L/ H = 0.76

   ∴ L = 0.76 x 40 = 30.4 ft.

   Summary: Use a nail strength ≈ 45.8^K and L = 30.4 ft.

7. Facing design (shotcrete)

   Moment @ nail $= \dfrac{0.49 T_{nail} L}{8}$ and moment between nails $= \dfrac{0.21TL}{8}$

   M_{max} = 0.49 (45.8^K)(5ft) / 8 = 14.0 k-ft x 0.9 = 12.63 k-ft

   (10% reduction due to soil support)

Shotcrete thickness: for 4ksi concrete $t = \sqrt{1.49M} = \sqrt{1.49(12.62)} = 4.33" + 2"\text{cover} = 6.33"$

   Area of steel required in facing: A_s (60ksi) :

   1.4 M = 0.9A_s(60ksi)(d-0.735A_s) →(1.4)(12.62)(12)=0.9A_s(60)(4.33-0.735A_s)

   Solving A_s = 4.77in^2 /lin ft. This is more than typical welded wire, hence rebar should be added to the welded wire mesh.

8. (alternate facing design, from Xanthakos,et.al.,1994)

   a. ACI code for 60ksi steel and 4ksi concrete: d = [0.2 TS]$^{1/2}$, where T = nail force, kips; S = nail spacing, ft, and d = shotcrete thickness, in.

   b. The A_s = 0.0052d (in^2)

   c. Accordingly, d = [0.2(45.8^K)(5ft)]$^{1/2}$ = 6.8 inches

   d. A_s = 0.0052 x 6.8” = .04 in^2

9. Punching Shear: (FHWA, 1996)

   $V_n(\text{kips}) = 0.126\sqrt{f_c'(\text{ksi})}\pi(D_c')(h_c)$; where D_c’ = bearing plate thickness + shotcrete thickness, and h_c = shotcrete thickness. Assuming a 12” x 12” bearing plate.

   $$V_n(\text{kips}) = 0.126\sqrt{4(\text{ksi})}\pi(6.33+12)(6.33) = 92\text{kips}$$

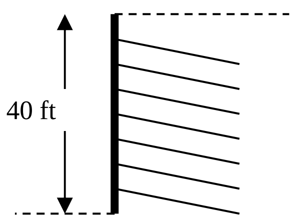
CALTRANS SNAIL COMPUTER PROGRAM (Xanthakos, 1994) (Caltrans, 2004)

Obviously due to arduous and time consuming calculations to solve a soil nailing design problem, one must resort to computer programs. The two programs available to FDOT are: SNAIL from CALTRANS, and GOLDNAL from Golder & Associates. SNAIL is a public-domain program that calculates global stability and individual nail stresses, and can be downloaded from www.dot.ca.gov/hq/esc/geotech/request.htm. Whereas GOLDNAIL is proprietary.

SNAIL Example

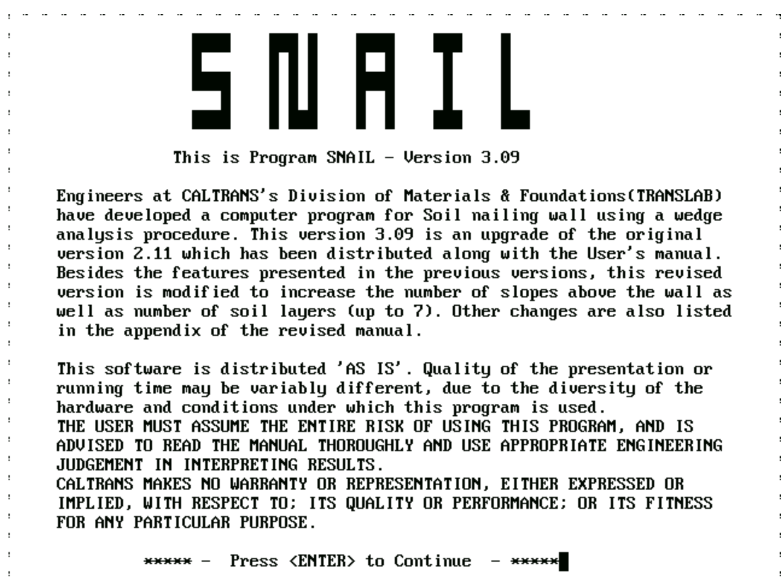
The following example is a comparison of the previous FHWA soil nail design charts illustration. The design is to determine: (a) nail length, (b) nail strength (diameter), and (c) facing thickness for a 40ft high wall. Nails are at 15° inclination.

Design FS = 1.2; i.e., $F_\phi = F_c = 1.2$



Soil Properties
γ = 120pcf
ϕ = 36° , c = 180 psf
f_l = 3000 psf
Nail Properties
$S_h = S_v$ = 5 ft
D_{grout} = 5 inches
D_{nail} = 1 inch , f_y =60ksi

USER'S GUIDE FOR SNAILZ



SNAIL

This is Program SNAIL - Version 3.09

Engineers at CALTRANS's Division of Materials & Foundations(TRANSLAB) have developed a computer program for Soil nailing wall using a wedge analysis procedure. This version 3.09 is an upgrade of the original version 2.11 which has been distributed along with the User's manual. Besides the features presented in the previous versions, this revised version is modified to increase the number of slopes above the wall as well as number of soil layers (up to 7). Other changes are also listed in the appendix of the revised manual.

This software is distributed 'AS IS'. Quality of the presentation or running time may be variably different, due to the diversity of the hardware and conditions under which this program is used.
THE USER MUST ASSUME THE ENTIRE RISK OF USING THIS PROGRAM, AND IS ADVISED TO READ THE MANUAL THOROUGHLY AND USE APPROPRIATE ENGINEERING JUDGEMENT IN INTERPRETING RESULTS.
CALTRANS MAKES NO WARRANTY OR REPRESENTATION, EITHER EXPRESSED OR IMPLIED, WITH RESPECT TO; ITS QUALITY OR PERFORMANCE; OR ITS FITNESS FOR ANY PARTICULAR PURPOSE.

***** - Press <ENTER> to Continue - *****

Opening screen of SNAIL. "Enter" to continue

```
*** ENGINEERING  UNIT ***

          INPUTS:      OUTPUTS:
 1. From  English  to  English measurement system.
 2. From  English  to  Metric  measurement system.
 3. From  Metric   to  Metric  measurement system.

Select: 1, 2, 3, or Default = 1 and hit ENTER    ?
```

Second screen.
Select "Units"

```
CALIFORNIA DEPARTMENT OF TRANSPORTATION
     ENGINEERING SERVICE CENTER
 DIVISION OF MATERIALS AND FOUNDATIONS

        SOIL REINFORCEMENT PROGRAM
          written by: OFFICE OF
      ROADWAY GEOTECHNICAL ENGINEERING
          5900 Folsom Boulevard
        Sacramento, California 95819

Project Title: fhwa h = 40ft L = 30ft

Would you like to change the title (Y/N or Default)?
```

Third Screen: Project Title information

```
  1.-WALL GEOMETRY:
 H = 40      ft-------Vertical Wall Height.
 B = 0       Degree---Wall Batter from Vertical Line.
 I1= 0      Degree| S1= 0      ft---1st Slope Angle and Distance.
 I2= 0      Degree| S2= 0      ft---2nd Slope Angle and Distance.
 I3= 0      Degree| S3= 0      ft---3rd Slope Angle and Distance.
 I4= 0      Degree| S4= 0      ft---4th Slope Angle and Distance.
 I5= 0      Degree| S5= 0      ft---5th Slope Angle and Distance.
 I6= 0      Degree| S6= 100    ft---6th Slope Angle and Distance.
 I7= 0      Degree-----7th Slope Angle.
  2.-REINFORCEMENT INPUTS:(Use OPTION 5 if LE, AL, SV, D, or BSF* varies.)
 N =   7     ---------Number of Reinforcement Levels.
 LE= 30      ft-------Reinforcement Length.
 AL= 15      Degree---Reinforcement Inclination.
SV1= 5       ft-------Vertical Distance to first Level.
 SV= 5       ft-------Vertical Spacing from second to N level.
 SH= 5       ft-------Horizontal Spacing
 PS= 92      Kips-----Punching Shear at reinforcement head.
 FY= 33      Ksi------Yield Stress of Reinforcement.
 D = 1       in-------Diameter of Reinforcement.
 DD= 5       in-------Diameter of Grouted Hole.
 Use Arrow and Return Keys to move around, Backspace and Delete Keys to edit
 When data entry finished, press Page Up, or Down, or Esc Key to Run program.
```

Fourth Screen

Wall height = 40 ft

I1 – I7 are backslope coordinates.
Since slope is horizontal I1–I7 = 0

N= 7 nails
Nail length = 30 ft
1st nail @ 5' from top
$S_v = S_H = 5$ ft
Punching Shear = 92k
$f_y' = 30$ ksi steel
Diam bar = 1"
Diam grout hole = 5"

```
  3.-SOIL PARAMETERS:
 NS =   1  Number of soil types.(1=Top layer to 7=Bottom layer-
           Layers must not intersect within limits of search).
      Weight| Angle|Cohes.| Bond*|  XS  |  YS  |  XE  |  YE
LAYER   Pcf | Deg. |  Psf |  Psi | (ft) | (ft) | (ft) | (ft)
  1    120     36    180    20.8
        0       0     0      0      0      0      0      0
        0       0     0      0      0      0      0      0
        0       0     0      0      0      0      0      0
        0       0     0      0      0      0      0      0
        0       0     0      0      0      0      0      0
        0       0     0      0      0      0      0      0
  4.-SEARCH LIMIT:
 LS= 0      ft-Begin Search.If LS=0, Search starts at wall crest.
 LN= 100    ft-End Search.(Horizontal Distance From Wall Toe).
+++++++++++ End of Data Inputs required to run SNAIL.++++++++++++
  5.-SURCHARGE: Maximum of 2 different surcharges are entered.
      First   | Second
 XL= 0        | 0        ft-----Begin Surcharge: Dist. from Toe.
 XR= 0        | 0        ft-----End Surcharge:   Dist. from Toe.
 PL= 0        | 0        psf/ft-Loading At Begin Surcharge.
 PR= 0        | 0        psf/ft-Loading At End Surcharge.
+++++ Use 'UP'or'DOWN'arrows to scroll. Hit 'Q' or 'q' to quit.+++++
```

Fifth Screen

Soil Properties

Note bond strength is p
psi: 3000 psf/144 = 20.8 psi

```
   6.-EARTHQUAKE ACCELERATION:
 KH= 0      A/G------Horizontal Earthquake Coefficient.
PKH= 0      %KH/100--Vertical Earthquake Coefficient.
   7.-WATER:
 FLAGW=  0  ==> 0= not Used. 1= Piezometric. 2= Phreatic
             1st Point     2nd Point     3rd Point
X-Coor.===> XW1=0     ft  XW2=0     ft  XW3=0     ft
Y-Coor.===> YW1=0     ft  YW2=0     ft  YW3=0     ft
 ************************ OPTION  #1 ******************************
 FLAGT= 0   ==> 0= Ultimate Bond, Yield, &Punching Shear values.
                1= Factored Bond, Yield, &Punching Shear values.
                2= Tie-back Wall only (with Soldier pile wall).
 ************************ OPTION  #2 ******************************
 FSEARCH= 0 ==> 0= The Search is Routinely from Nodes 1 to 10.
                1= The Search is conducted from nodes LA to LB.
                2= For Specified Failure Plane. Input II And JJ.
    LA= 1   Beginning at node 'LA'.        II = 0     Horizontal
    LB= 10  Ending at node 'LB'.           JJ = 0     Vertical
 ************************ OPTION  #3 ******************************
 FLAG =  0  ==> 0= There is no TOE; 1= There is TOE. Enter DATA:
 1st Slope Angle|1st Slope Length| 2nd Slope Angle|2nd Slope Length
 I8= 0    Degree| S8= 0      Feet| I9= 0    Degree| S9= 0      Feet
 SD= 0   Ft, Vertical Depth of search.| NTS= 0   No. of Searches.
```

Sixth Screen

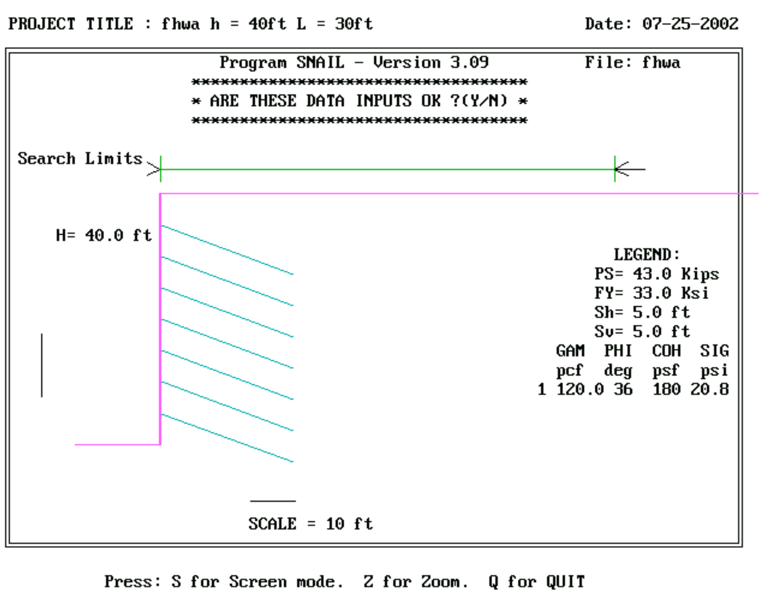
```
************************ OPTION  #4 ******************************
PD= 0     Kips/ft-Width. External force on Wall. -->(+)|(-)<--
AN= 0     Degrees from horizontal. Positive = Counterclockwise.
************************ OPTION  #5 ******************************
FLAGN=  0  ==> 0= OPTION #5 is not Used; 1= Used. Enter DATA:
  Reinf.       Reinf.       Vert.        Bar          Bond
 Length      Inclination   Spacing     Diameter      Stress
  (ft)        (Degree)      (ft)        (inch)       Factor*
LE(01)= 0     AL(01)= 0    SV(01)= 0    D(01)= 0     SIG(01)= 1
LE(02)= 0     AL(02)= 0    SV(02)= 0    D(02)= 0     SIG(02)= 1
LE(03)= 0     AL(03)= 0    SV(03)= 0    D(03)= 0     SIG(03)= 1
LE(04)= 0     AL(04)= 0    SV(04)= 0    D(04)= 0     SIG(04)= 1
LE(05)= 0     AL(05)= 0    SV(05)= 0    D(05)= 0     SIG(05)= 1
LE(06)= 0     AL(06)= 0    SV(06)= 0    D(06)= 0     SIG(06)= 1
LE(07)= 0     AL(07)= 0    SV(07)= 0    D(07)= 0     SIG(07)= 1
LE(08)= 0     AL(08)= 0    SV(08)= 0    D(08)= 0     SIG(08)= 0
LE(09)= 0     AL(09)= 0    SV(09)= 0    D(09)= 0     SIG(09)= 0
LE(10)= 0     AL(10)= 0    SV(10)= 0    D(10)= 0     SIG(10)= 0

*NOTES: The Bond Stress Factor (BSF) is a mutiplier of Bond applied
 throughout a bar, regardless of soil parameters. (Default = 1.00)

+++++ Use 'UP'or'DOWN'arrows to scroll. Hit 'Q' or 'q' to quit.+++++
```

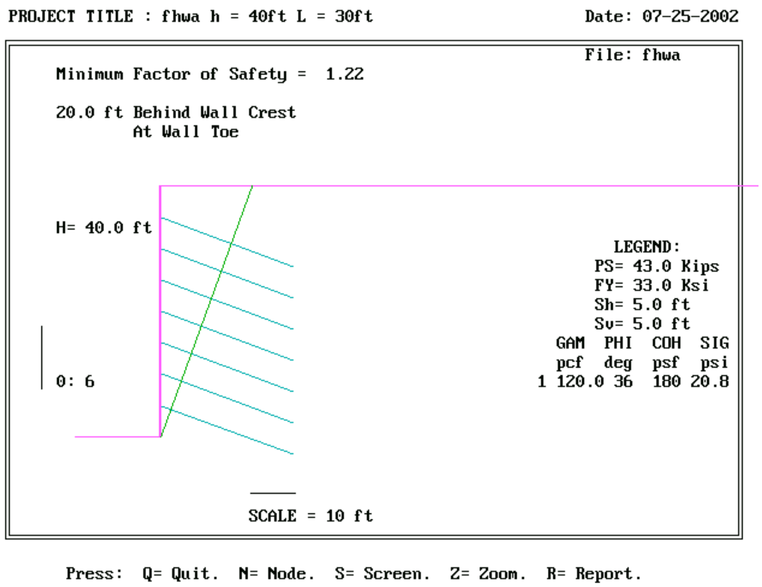
Seventh Screen

Option 5 allows "customizing" each nail layer.



Eighth Screen

Presents view of problem prior to "run"

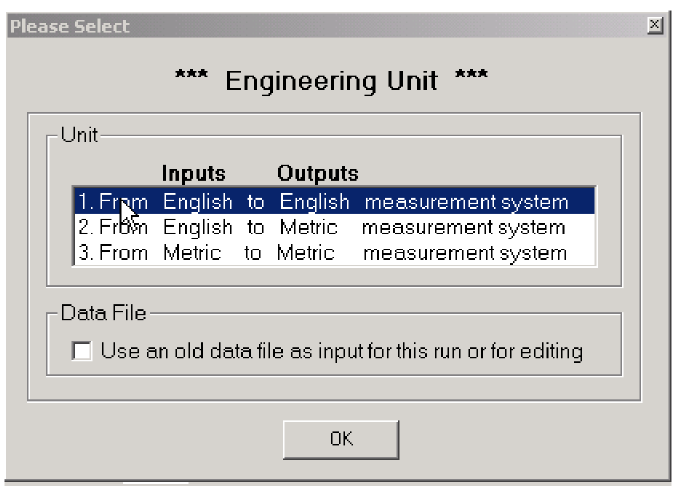


Ninth Screen

Presents results

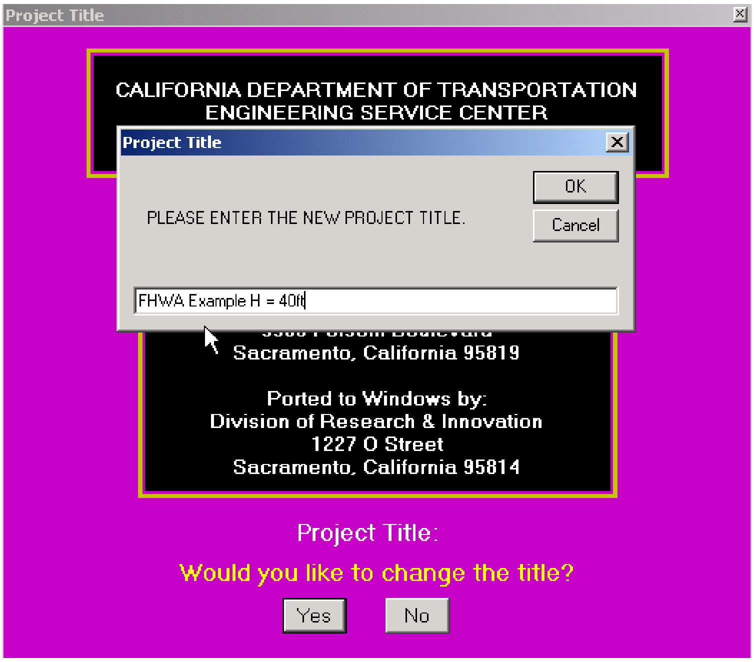
Note FS = 1.22 which agrees with hand calculation example problem

USER'S GUIDE FOR SNAIL-WIN



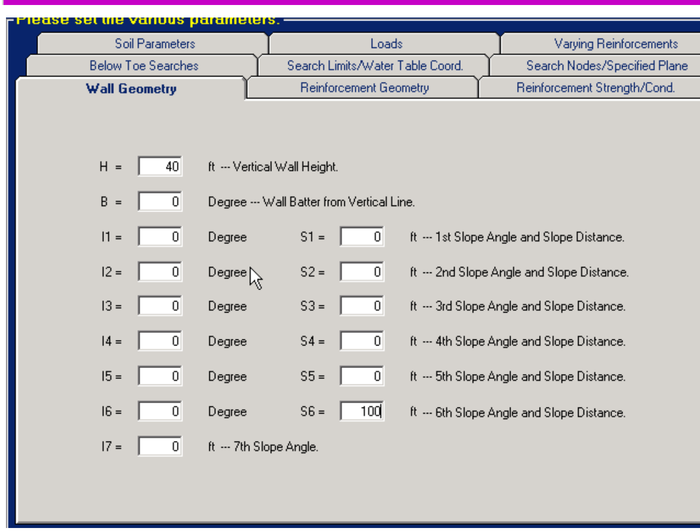
Opening screen, select units, and

Unclick "use old data file"



Second screen

Enter "title"



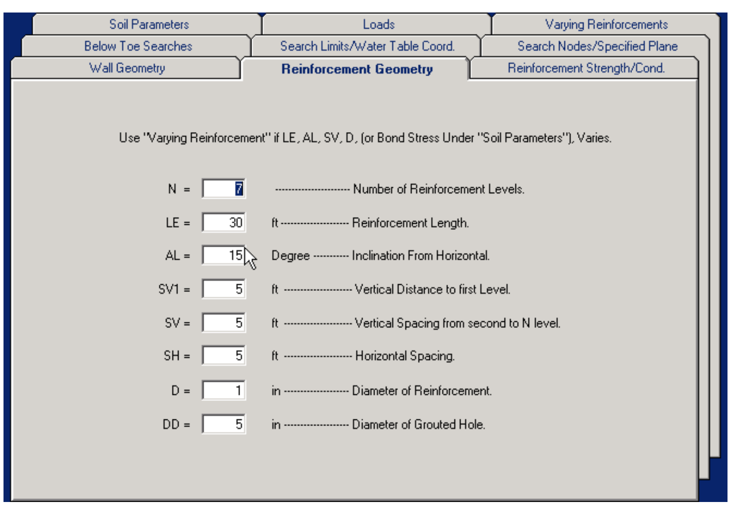
Third Screen

Use tool bar lower left to get Wall Geometry

Enter H = 40 ft,

I1-I7 are backslope coordinates. Since slope is Horizontal, I1-I7 = 0

S6 = 100 ft



Fourth Screen

Use tool bar lower left

Reinforcement metry

Enter N = 7 layers, L = 30 ft

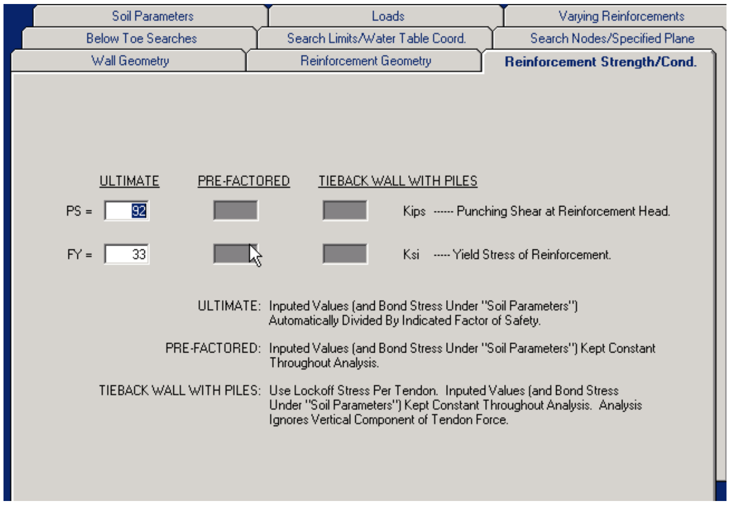
Inclination = 15°, 1st nail @ 5' from top

S_{V1} = 5 ft

S_V = 5 ft, S_H = 5 ft

Diam steel rod = 1 in.

Diam grout hole = 5 in.

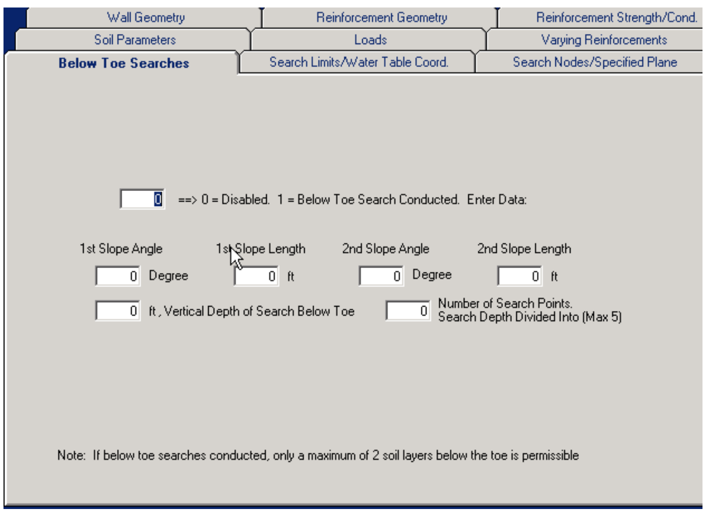


Fifth Screen

Use lower left tool bar to get Reinforcement Strength cond.

Enter: Punching shear = 92 k.

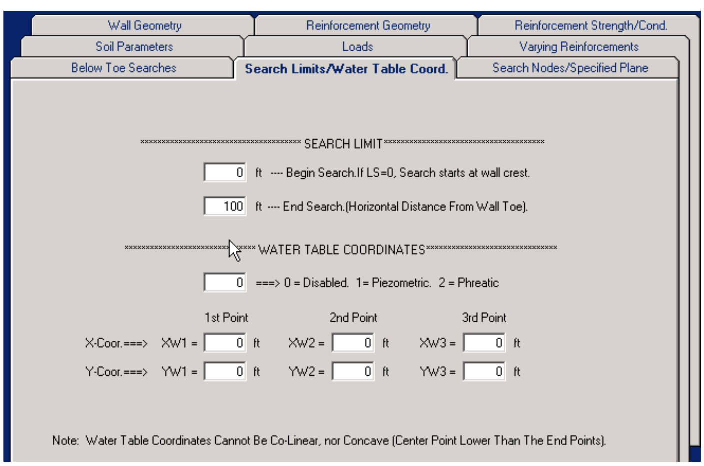
Yield stress = 33 ksi



Sixth Screen

Use lower left tool bar to get "Below Toe Searches"

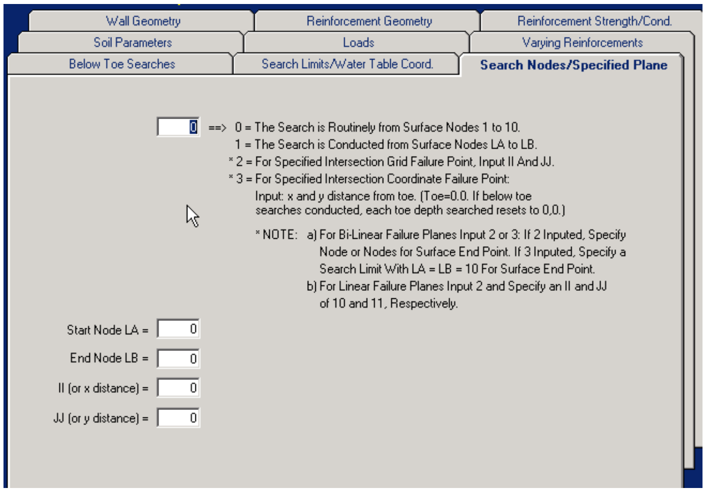
Enter 0 = disabled



Seventh Screen

Use lower left toolbar to get Search Limits

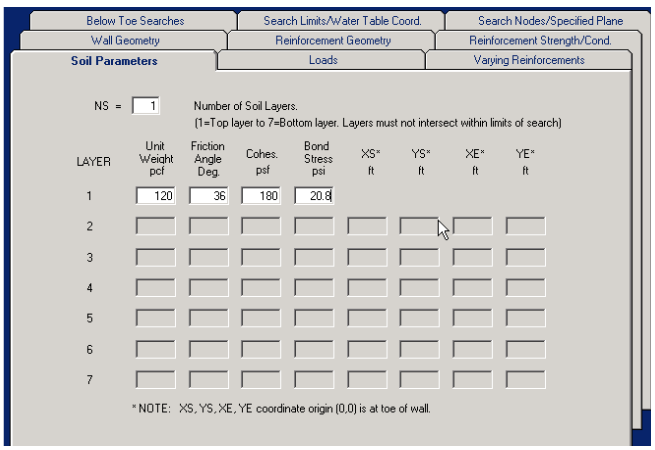
Enter: begin = 0, end = 100



Eighth Screen

Use lower left tool bar to get "Search Modes"

Select 0, search nodes 1 to 10



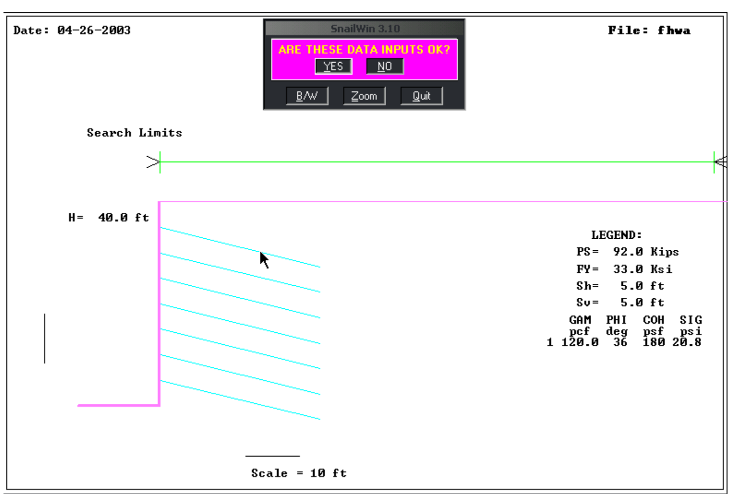
Ninth Screen

Use lower left tool bar to get "Soil Parameters"

Enter: No. soil layers = 1
γ = 120 pcf, ϕ = 36°, c = 180 psf
bond stress = 3000 psf

3000/144 = 20.8 psi

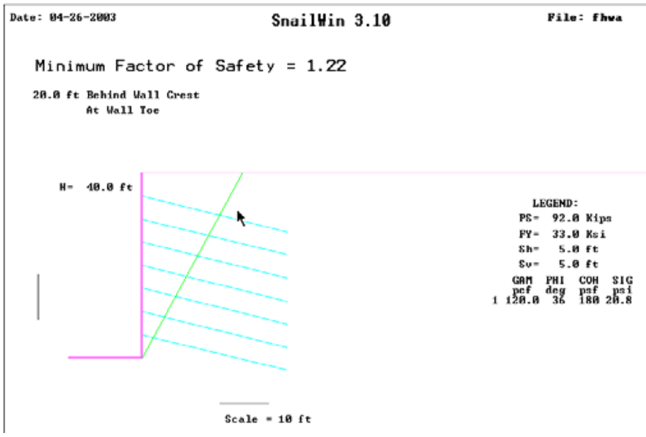
Now "**Execute**"



Tenth Screen

Graphical display of input
Select "OK"

Program "runs"



Eleventh Screen – Output

FS = 1.22 Agrees !

SPECIFICATIONS (from FHWA, 1996)

Contracting Methods and Definitions

The general types of contracting methods currently being used for soil nail wall design and construction may be generally classified as owner Design and Contractor Design/Build. Owner Design Contracts may be further structured using an Owner Design – Performance or an Owner Design – Procedural/Prescriptive based specification. Design/build contracting methods are performance based and place the responsibility of both design and construction on the contractor. However, the procedural/prescriptive based method is not recommended since the owner is fully responsible for the design and performance, as well as directing the contractor's work if changes are needed.

Owner Design – Performance Specification

Nail final drillhole diameter and installation method required to provide the design nail pullout resistance is the contractor's responsibility.

Owner Design – Procedural/Prescriptive Specification

Nail drillhole diameter and installation method is specified by owner.

Design/Build – Performance Specification

Implicit – Owner determines that a soil nailing wall is feasible and specifies a soil nailing wall. The contractor prepares the design calculations and detailed plans and constructs the wall.

Open – Owner specifies that a wall be built. The contractor selects the wall type, prepares the design calculations and detailed plans, and constructs the wall.

The advantages and disadvantages of these various methods are summarized in the following tables.

Owner Design – Performance Specification

Advantages	Disadvantages
Valuable in-house expertise is obtained	Requires owner staff and expertise
Owner has control over final product	Requires sufficient staff to support project
Equitable risk is shared between the owner and the contractor	Requires assumptions regarding the contractor's construction procedures and equipment
Contractor's experience, equipment, and expertise is utilized	Less economical if the design does not optimize the contractor's procedures and equipment

Owner Design – Procedural/Prescriptive Specification

Advantages	Disadvantages
Widens the bidding field	Owner assumes all risks
	Owner if fully responsible for the design and performance of the system
	Owner directs the contractor's work when changes are required
	Owner must be highly confident in predicting the contractor's performance
	The owner must have highly qualified and experienced design and inspection staff
	Unqualified contractors may be awarded the contract
	Potential for claims and cost overruns is high

The procedural ("prescriptive") contracting method is not generally advantageous for soil nail wall construction and is not recommended.

Contractor Design/Build

Advantages	Disadvantages
Cost effective	Owner assumes maintenance responsibilities if the structure is not warranted
May be advantageous when very difficult ground is expected	Owner has less control over the design unless pre-bid design is used.
Dose not require large owner staff	Potential for undesirable or unfamiliar design features to be incorporated into the design
Requires less in-house expertise than required for owner design	Owner must still provide inspection to assure construction quality is acceptable
Design is tailored to the contractors con-struction procedures and equipment	Requires adequate in-house expertise to review design, submissions and monitor construction operations
Provide incentive for contractor innovation	
Allows contractor to use proprietary knowledge and methods	

Guide specifications are presented in Appendix B-Permanent Soil Nails and Wall excavation guide, and Appendix C Shotcrete and wall drainage guide of FHWA (1996)

REFERENCES

ASCE Geotechnical Special Publication No. 69 (1997). “Ground Improvement, Ground Reinforcement, Ground Treatment Development 1987 – 1997.” Edited by R. Schaefer.

Bang, S. (1991), Contribution to Summary of Design Methods Comparison for Nailed Retaining Walls, FHWA Demonstration Project 82.

Bridle, R. and Myles, B. (1991), A Machine for Soil Nailing Process and Design, ECOLE Nationale des Ponts et Chaussees, Civil Engineering European Courses in Soil Reinforcement, Paris.

Byrne, J. (1991), Contribution to Summary of Design Methods Comparison for Nailed Retaining Walls, FHWA Demonstration Project 82.

CALTRANS (2004) SNAILWIN http://www.dot.ca.gov/hq/esc/geotech/requests/snailwin11042002.zip

CALTRANS (1991), "A User's Manual for the SNAIL Program, Version 2.02 - Updated PC Version," Obtained from Mr. K.A. Jackson, California Department of Transportation, Division of New Technology, Material & Research, Office of Geotechnical Engineering, Sacramento, Oct.

Chassie, R.G., 1994, "FHWA Ground Nailing Demonstration Project, Guideline Manual and Workshop," United States Federal Highway Administration Publication.

Choukeir, M.H. (1996) Seismic Analysis of Reinforced Earth and Soil Nailed Structures, Dissertation for PhD, Polytechnic University, New York.

Christopher, B.R. et al., (1990), 'Reinforced Soil Structures Vol. 1 - Design and Construction Guidelines," US Federal Highway Administration, Publication No. FHWA-RD-89-043.

Elias, V. and Juran, I. (1991), "Soil Nailing for Stabilization of Highway Slopes and Excavations," United States Federal Highway Administration, Publication No. FHWA-RD-89-193, June.

Felio, G.Y., Vucetic, M., Hudson, M., Barar, P., and Chapman, R., 1990, “Performance of Soil Nailed Walls During the October 17, 1989 Loma Prieta Earthquake,” IN *Proc. of the 43rd Canadian Geotechnical Conference, Quebec, Canada*, pp. 165-173.

FHWA (1998) “Ground Improvement Technical Summaries – Vol I and Vol II – Demonstration Project 116” by Elias, V., Welsh, J., Warren, J., Lukas, R. FHWA Publication No. FHWA-SA-98-086.

FHWA (1996) “Manual for Design and Construction Monitoring of Soil Nail Walls” Report No. FHWA-SA-96-069, Authors: Byrne, R.J., Cotton, D., Porterfield, J., Wolschlag, C., Ueblacker, G. Report available from: http://www.fhwa.dot.gov/bridge/geopub.htm

French National Research Project CLOUTERRE (1991) "Recommendations CLOUTERRE 1991 - Soil Nailing Recommendations Presses de l'Ecole Nationale des Ponts et Claussees" English Translation – FHWA-SA-93-026.

Goldstein, N. (2001) “Soil Nailing” Erosion Control, Sept/Oct Issue.

ISSMFE-TC-17, “Soil Nailing” http://tc17.poly.edu/sn.htm, CEN/TC/288/WG9/68E CEN Technical Committee 288 Working Group 9 Reinforced Soil.

Ingold, T.S. (2000) “Draft European Standard Execution of Special Geotechnical Work - Soil Nailing”, CEN/TC/288/WG9/68E CEN Technical Committee 288 Working Group 9 Reinforced Soil http://www.personal.dundee.ac.uk/~amcjones/cen/fd/contents.htm

Ingold, T.S. and Myles, B. (1996), Ballistic soil nailing, Earth Reinforcement, H. Ochiai, N. Yasufuku, and K. Omine, eds., *Proceedings of the International Symposium on Earth Reinforcement*, Japan.

Juran, I., and Elias, V. (1987), Soil nailed retaining structures: Analysis of case histories, Soil Improvement - A Ten Year Update, *ASCE Special Geotechnical Publication* No. 12, Ed. JP Welsh, pp. 232-244.

Juran, I., Baudrand, G., Farrag, K. and Elias, V. (1988) Kinematical limit analysis for design of nailed structures, Journal of Geotechnical Engineering, ASCE, Vol. 116, No. 1, pp. 54-72.

Juran, I. and Elias, V. (1991) Ground Anchors and Soil Nails in Retaining Structures, *Foundation Engineering Handbook*, 2nd Edition, pp. 868-905.

Louis, C. (1986), Theory and practice in soil nailing temporary or permanent works, *ASCE Annual Conference*, Boston.

Mitchell, J.K., et al. (1987), Reinforcement of earth slopes and embankments, *National Cooperative Highway Research Program Report* No. 290, Transportation Research Board, June.

Peck, R.B., 1969, “Deep Excavations and Tunnelling in Soft Ground,” IN *Proc. 7th Int. Conf. Soil Mech. Found. Eng., Mexico City.*

Plumelle, C. (1993), Full scale test of a soil nailed wall: design, construction and experimentation, "Soil reinforcement full scale experiments" pp. 35-62.

Plumelle, C. (1993), Conception, execution et experimentation d'un massif en sol cloue en vraie grandeur, C.R. Symposium International sur le Renforcement des Sols: Experimentations des annees 1980, ENPC, Paris, pp. 35-62.

Plumelle, C. Schlosser, F., Oclage, P. and Knochenmus, G. (1990), French National Research Project on Soil Nailing: CLOUTERRE, Geotechnical Special Publication No. 25, ASCE, pp. 660-675.

Schlosser, F., Unterreiner, P., and Plumelle, C. (1993) Validation des methodes de calcul de clouage par les experimentations du Projet National CLOUTERRE. Revue Francaise de Geotechnique, 64.

Shen, C.K., (1991), Contribution to Summary of Design Methods Comparison for Nailed Retaining Walls, FHWA Demonstration Project 82.

Shen, C.K., Bang, S. Romstad, J.M., Kulchin, L., and Denatale, J.S. (1981b), Field measurements of an earth support system, *Journal of the Geotechnical Engineering Division, ASCE*, **107**, No. GT-12.

Stocker, M.F., Korber, G.W., Gassler, G., and Gudehus, G. (1979), Soil nailing, *International Conference on Soil Reinforcement*, Paris, **2**, pp. 463-474.

Thompson, S.R. and I.R. Miller, (1990), "Design, Construction and Performance of a Soil Nailed Wall in Seattle, Washington," Design and Performance of Earth Retaining Structures, P.C. Lambe and L.A. Hansen, Eds., ASCE Geotechnical Special Publication No. 25, New York, pp. 629-643.

Welsh, J.P. et al. (1987), Soil improvement - A ten year update, *ASCE Geotechnical Special Publication* No. 12.

Xanthakos, P., Abramson, L.W., and Bruce, D.A. (1994), In situ ground reinforcement, *Ground Control and Improvement*, J. Wiley Publishers p. 381

Chapter 13

GEOTEXTILES IN ROADWAY APPLICATIONS

INTRODUCTION

Although geotextiles have been used in temporary low volume roads (haul roads, logging, etc.) this chapter's discussion is devoted to the use of geotextiles in permanent high volume paved roads. Accordingly, the role of the geotextile is: (1) separation of base course from subbase and subgrade materials, and (2) reinforcement.

Separation

This is accomplished by preventing migration of the subgrade fines into the base course aggregate and migration of the base-course aggregate into the subgrade. This mixing of the base course aggregates would cause deterioration of the base-course structural properties and subsequent roadway capacity. Figure 13.1 illustrates this concept. However, as a successful geotextile, the geotextile must function successfully as a filter, yet provide sufficient drainage.

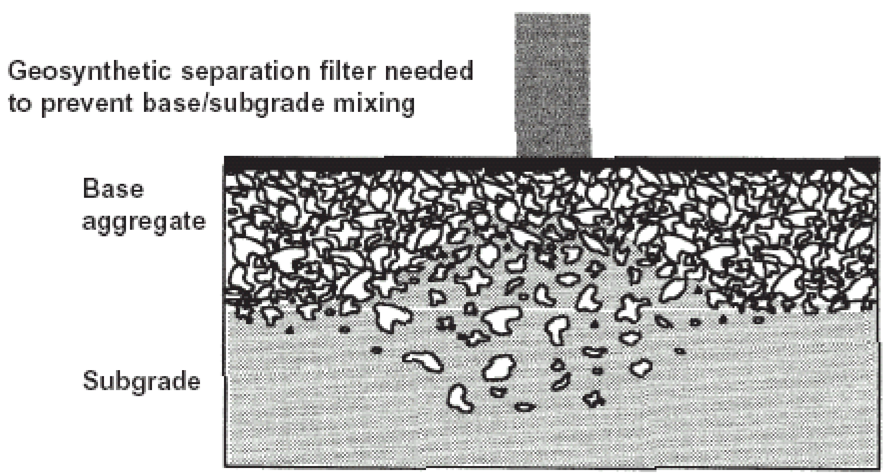


Figure 13.1 Illustration of Roadway Geotextile Separator (from Tensar)

Water Flow Through Geotextiles

Darcy's Law governs the flow of water through soils and geotextiles; as:

$Q = kiA$; where Q = flow through unit area per unit time, ft^3/s;

A = cross-sectional area, ft^2 ;

k = coefficient of permeability, ft/s;

i = dh/dx = hydraulic gradient, h = head, x = distance.

However, in the case of geotextiles, dx in Darcy's Law represents the fabric thickness, which is quite small and difficult to measure. This has led to the definition of permissivity, ψ, for flow perpendicular to the fabric; or:

$$Q = \frac{k}{dx} dhA = \Psi dhA$$

where $\Psi = \frac{k}{dx} = \text{permissivity}(s^{-1})$ and permissivity is simply the coefficient of permeability divided by the geotextile thickness.

For drainage applications, the flow rate of water through or transmitted within the geotextile is important. Considering, the cross-sectional area, A, of a geotextile strip with width (a) and thickness (b) A = ab

Then, Darcy's Law can be written to define, transmissivity, θ.

$$Q = (kb)ta = \theta\ ta, \text{ where } \theta = kb = \text{transmissivity } (l^2/s)$$

Transmissivity is the quantity of water that flows within the geotextile strip of unit width under a unit gradient.

Filtration

The function of a geotextile filter is to retain the soil while allowing the liquid to flow as freely as possible. In order to achieve this objective, a geotextile filter needs to meet: (1) **Retention criterion**: the filter opening size must be sufficiently small to retain soil particles. (2) **Permeability criterion**: the filter must be sufficiently permeable to ensure that the liquid flow is as free as possible, and (3) **Porosity criterion**: the filter should remain a high porosity so the probability for clogging is small. (LANDFILLDESIGN.com).

The pore size distribution of geotextiles can be determined by sieving glass beads of a known size through the geotextile. Successively coarser beads are used until 5% or less are passing through the geotextile. The apparent opening size (AOS) is defined as the particle size of the glass bead corresponding 5% passing or 95% (O_{95}) retained. For example, a O_{95} = 0.25 mm means 95% of glass beads with a diameter of 0.25 mm are retained by the geotextile. The AOS size is conveniently given by geotextile manufacturers spec. sheets.

AASHTO M288-96 recommends the criteria shown in Table 13.1 for the selection of a geotextile filter.

Table 13.1 Geotextile Criteria for Subsurface Drainage
(after AASHTO M288-96)

Filter Criteria	Percent Soil Passing No. 200 (0.075 mm) Sieve		
	< 15	**15 – 50**	**> 50**
Minimum Permissivity, ASTM D-4491	0.5 sec^{-1}	0.2 sec^{-1}	0.1 sec^{-1}
Maximum AOS, ASTM D-4751	0.43 mm	0.25 mm	0.22 mm

Alternatively, Giroud's filter criteria can be used for geotextile filter design. Giroud (2000) uses a linearization of the particle distribution curve that, when plotted with the classical log scale horizontal axis, is as close as possible to the actual particle distribution curve (Figure 13.2). It should be noted in Figure 13.2 that there is greater uncertainty on the two extremities (d_0'? and d_{100}) of the actual particle size distribution. This justifies the use of the linear particle size distribution curve. The result obtained using Giroud's retention criterion is not affected by the truncation of the particle size distribution curve. Tables 13.2 and 13.3 present Giroud's criteria. (However, I prefer Table 13.3.)

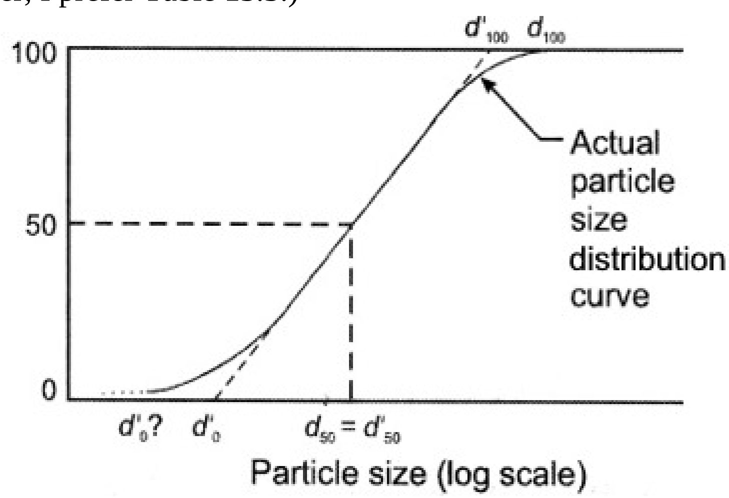


Figure 13.2 Linearization of Particle Size Distribution Curve (after Giroud, 2000)

Table 13.2 Retention Criterion for the Hyperstable Case (C_{cu}' = 3) Expressed Using d_{85S}'

Soil Density	Density Index (Relative Density) I_D	Relative Compaction (R_C)	Linear Coefficient of Uniformity of the Soil C_u'	
			$1 \le C_u' \le 3$	**$C_u' > 3$**
loose	$I_D \le 35\%$	$R_C \le 86\%$	$O_F \le (C_u')^{0.3} d_{85S}'$	$O_F \le (9/C_u'^{1.7})\, d_{85S}'$
medium dense	$35\% \le I_D \le 65\%$	$86\% \le R_C \le 92\%$	$O_F \le 1.5\,(C_u')^{0.3} d_{85S}'$	$O_F \le (13.5/C_u'^{1.7})\, d_{85S}'$
dense	$I_D > 65\%$	$R_C > 92\%$	$O_F \le 2\,(C_u')^{0.3} d_{85S}'$	$O_F \le (18/C_u'^{1.7})\, d_{85S}'$

Table 13.3 Retention Criterion for the Hyperstable Case ($C_{cu}' = 3$) Expressed Using d_{50S}'

Soil Density	Density Index (Relative Density) I_D	Relative Compaction (R_C)	Linear Coefficient of Uniformity of the Soil C_u'	
			$1 \le C_u' \le 3$	$C_u' > 3$
loose	$I_D \le 35\%$	$R_C \le 86\%$	$O_F \le C_u' \, d_{50S}'$	$O_F \le (9/C_u') \, d_{50S}'$
medium dense	$35\% \le I_D \le 65\%$	$86\% \le R_C \le 92\%$	$O_F \le 1.5 \, (C_u') \, d_{50S}'$	$O_F \le (13.5/C_u') \, d_{50S}'$
dense	$I_D > 65\%$	$R_C > 92\%$	$O_F \le 2 \, (C_u') \, d_{50S}'$	$O_F \le (18/C_u') \, d_{50S}'$

where:

C_u' linear coefficient of uniformity of the soil = d_{60S}'/d_{10S}'

I_D relative density or density index of the soil

d_{ms}' the particle size such that m% (on the linear particle size distribution curve) of the linear soil particles by mass are smaller than d_{ms}'

R_C relative compaction

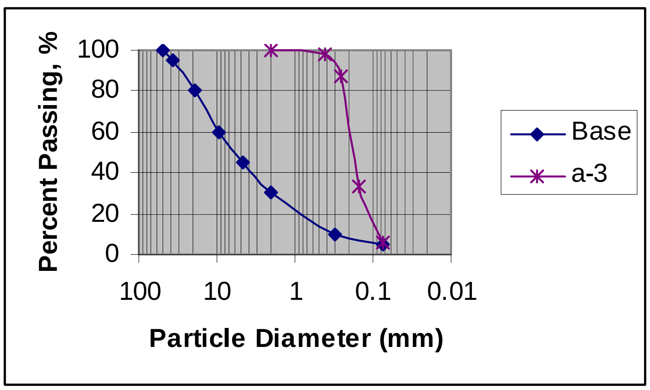
O_F maximum filter opening size = O_{95} or AOS

For fine-grained materials containing more than 10% fines, and a PI> 5, O_{95} should be < 0.21mm.

Permeability Criteria – The goetextile permeability should be greater than that of the retained soil. This criteria is easily met for fine-grained retained soils.

Porosity Criteria - There are two mechanisms that are known to cause progressive clogging in a filter: (1) Chemical, biological and biochemical clogging; (2) Accumulation of soil particles on or in the filter. Porosity criteria is often met with nonwoven geotextiles since the typical porosity value for a nonwoven geotextile is 0.7 - 0.9 (uncompressed) or 0.5 (compressed). However, there are still woven geotextiles with a porosity of only 4%. Hence, $N_{GTX} > 0.3$ where N_{GTX} is the porosity of geotextile filter.

Example: Consider a granular base course overlying a compacted SM-SP sub-base (A-3), between which a geotextile is to be placed as a separator/filter. The gradation of the base and sub-base are shown:



From the gradation curves, the following $D_{\%}$ sizes are obtained:

Diameter % $D_{\%}$	Base Course, mm	Sub-Base, mm
D_{10}	0.3	0.095
D_{20}	0.9	0.11
D_{50}	6.0	0.19
D_{60}	9.5	0.20
D_{85}	25	0.25
$C_u = D_{60} \div D_{10}$	9.5÷0.3 = 31.67	0.2÷ .095 = 2.11

Using Landfilldesign.com, Giroud's (2000) linearized grain size curve results in:

Diameter % $D_{\%}$	Linear Base Course, mm	Linear Sub-Base, mm
D_0'	0.375	0.088
D_{10}'	0.603	0.10
D_{20}'	1.019	0.115
D_{50}'	4.916	0.171
D_{60}'	8.305	0.195
D_{85}'	30.811	0.273
D_{100}'	67.658	0.333
R^2	0.99	0.972
$C_u' = D_{60}' \div D_{10}'$	8.305 ÷ 0.603 = 13.77	0.195 ÷ 0.10 = 1.95

Using criteria from Table 13.3, the O_{95} AOS values can be calculated:

Soil	Table 3 Criteria	O_{95} AOS, mm
Base Course	$C_u' > 3$, $O_{95} \leq (18/C_u')D'_{50}$	$O_{95} \leq (18/13.77)(4.916) = 6.43$ mm
Sub-Base	$1 \leq C_u' \leq 3$, $O_{95} \leq 2.0(C_u')D_{50}'$	$O_{95} \leq 2.0\ (1.95)\ (.171) = 0.67$ mm

From "View Material to these Design Specs" @ Landfilldesign.com, the following example selections result:

Manufacturer	Product	Polymer Type [2]	Structure [1]	Mass/Unit Area ASTM D 5261 g/m^2	Other MSA [8]	M288 Survivability Class	M288 Applications [4]	
Percent Open Area CWO-22125 %	Apparent Opening Size ASTM D 4751 mm	Permittivity ASTM D 4491 sec^{-1}	Flow Rate l/min/m^2	Test Method [3]	Puncture ASTM D 4833 kN	Trap Tear ASTM D 4533 kN	Grab Tensile ASTM D 4632 kN	Elongation ASTM D 4632 kN
Wide Width Tensile Strength @ 5% Strain - MD ASTM D 4595 kN/m	Wide Width Tensile Strength @ 5% Strain - XD ASTM D 4595 kN/m	Wide Width Tensile Ultimate Strength - MD ASTM D 4595 kN/m	Wide Width Tensile Ultimate Strength - XD ASTM D 4595 kN/m	Creep Limited Strength ASTM D 5262 kN/m [6]	T_{allow} GRI GT7 (in sand) [7]			
Amoco Fabrics & Fibers Co.	Amoco 4535	PP	NW-P		F, D, SP		NP	
	0.212	2.2	6510	FH	0.2	0.155	.355	50
Amoco Fabrics & Fibers Co.	Amoco 1198	PP	W		E, D, F, SP, ST, SF	3	D	
	0.425	0.5	1420	FH	0.53	0.285	1.33	15

[1] Structure

NW = Nonwoven NW-P = Nonwoven Needlepunched NW-H = Calendared

W = Woven W-SF = Woven Slit Film O/C = Other/Combination

[2] Polymer Type

PP = Polypropylene PET = Polyester

[3] Test Method

FH = Falling Head CH = Constant Head

[4] M288 Applications

SP = Separation ST = Stabilization S/F = Filt Fence D = Drainage

F = Filtration E = Erosion Control A/O = Asphalt Overlay

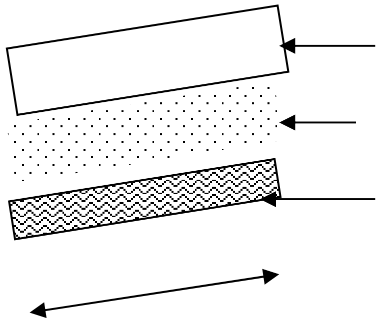
[6] For a minimum of 10,000 hours, extrapolated to a 75-year time period.

Both Amoco 4535 and 1198, have AOS values < 6.43 (base) and 0.67 (sub-base) and thus would be acceptable. However, AASHTO criteria Table 13.1 recommends for < 15 % fines, that the maximum AOS = 0.43. Consequently, Amoco 4535 is unsuitable and Amoco 1198 is marginal. AASHTO also recommends that permissivity be > 0.5 sec^{-1}, for which Amoco 4535 is unsuitable and Amoco 1198 is marginal.

Drainage-

Use of geotextiles for drainage involves calculating geotextile transmissivity criteria based upon infiltration rates. The following example illustrates the logic.

Drainage example (Huang, 1993): Evaluate the use of Amoco 1198 as a drainage layer when placed between the 6-inch base and sub-base of a 24 ft. wide two-lane PCC pavement having a 2% slope.



1. Estimate infiltration rate of water: Cedergren (1973) recommends multiplying the 1-hr design rainfall intensity by a coefficient of 0.33 to 0.50 for asphalt and 0.50 to 0.67 for concrete pavements.

For this problem, Florida design rainfall = 2 inches/hr. Hence q_i = 0.5 (2 in/hr) = 1"/hr.

Thus, $q = q_i W_p = \frac{1"}{hr} \times \frac{1ft}{12"} \times 24ft \times \frac{1ft}{1ft} = \frac{2ft^3}{hr \cdot ft} = \frac{48ft^3}{day \cdot ft}$

2. Estimate the permeability of base: Moulton (1980) uses:

$k(ft/d) = \frac{6.124 \times 10^5 (D_{10})^{1.478} (n)^{6.654}}{(P_{200})^{0.597}}$, where D_{10} is effective size in mm, n = porosity

and found by $n = 1 - \frac{\gamma_d}{62.4 G_s}$, γ_d = dry unit weight, pcf, and P_{200} is the percent fines.

For this problem using the base course grain size D_{10} = 0.3mm, and P_{200} = 5 %. Estimating a unit weight = 120 pcf, results in: n = 1-120/(62.4 x 2.7) = 0.29

$$k = \frac{6.214 \times 10^5 (0.3)^{1.478} (0.29)^{6.654}}{5^{0.597}} = 10.62 ft/d$$

3. Estimate the steady-state capacity of the drainage layer without the geotextile (Baber and Sawyer, 1952)

$q = kH\left(S + \frac{H}{2L}\right)$, where q = discharge capacity, k = permeability, S = slope, H = layer thickness, and L = length of drainage layer.

For this problem,

$$q = 10.62\text{fpd} \times (0.5\text{ft})\left(0.02 + \frac{0.5\text{ft}}{2 \times 24\text{ft}}\right) \times (1\text{ft unit thickness}) = 0.16\text{ft}^3/\text{d}/\text{ft}$$

But infiltration is $\frac{48\ \text{ft}^3}{\text{day} \cdot \text{ft}}$, which is > 0.16 ft^3/d/ft, so the layer is insufficient for steady state flow.

1. Estimate the steady-state capacity of the drainage layer including the geotextile. The flow rate of Amoco 1198 is

$$q = \frac{1420\text{l}}{\text{min} \cdot \text{m}^2} \times \frac{0.264\text{gal}}{\text{l}} \times \frac{1\text{ft}^3}{7.5\text{gal}} \times \frac{1440\,\text{min}}{\text{day}} \times \frac{1\text{m}^2}{(3.3)^2\,\text{ft}^2} = 6609\,\frac{\text{ft}^3}{\text{day} \cdot \text{ft}^2}$$

And the permeability for the geotextile can be represented as

$$k = \theta\ i = \frac{6609\text{ft}^3}{\text{d} \cdot \text{ft}^2} \times 0.02 = 132.2\text{ft}/\text{day}$$

Combining permeabilities by assuming the geotextile thickness is 0.15”:

$$k_{eq} = \frac{\sum k_i h_i}{\sum h_i} = \frac{(10.62'/\text{d})(0.5) + (132.2'/\text{d})(0.15/12)}{0.5' + 0.15''/12)} = 13.6\text{ft}/\text{d}$$

Thus the combined capacity of the base and geotextile is:

$q = (13.6'/\text{d}) \times (0.5') \times (0.02 + \frac{0.51}{2(24)} = \frac{0.21\text{ft}^3}{\text{d} \cdot \text{ft}}$, which is still considerably less than the infiltration of 48 ft^3/d/ft.

2. Estimate the unsteady state flow: Casagrande and Shannon (1952) showed that the time for 50% drainage can be estimated as;

$t_{50} = \frac{nL^2}{2k(H + SL)}$, where: t_{50} = time for 50% drainage, n = porosity, L = length of drainage layer, S = slope, and H = drainage layer thickness.

For conditions without the geotextile, $t_{50} = \frac{0.29(24)^2}{2(10.62)(0.5 + (.02 \times 24)} = 8^d$ Fair

For conditions with the geotextile, $t_{50} = \frac{0.29(24)^2}{2(13.6)(0.5 + (.02 \times 24)} = 6.3^d$ Good

Invoking AASHTO (1986) criteria

Rating	Water removed within
Excellent	2 hours
Good	1 day
Fair	1 week
Poor	1 month
Very Poor	Never drain

Pavement Reinforcement Using Geogrids (Tensar)

Base course reinforcement is analogous to reinforced concrete, whereby instead of steel rebar being used to provide tensile resistance, high strength plastic (HDPE) geogrids are used to provide tensile resistance to the cohesionless base materials; thereby restraining aggregate movement. The benefit being: reduced rutting due to lateral movement of the base course aggregate, reduced maintenance, and reduced base course thickness as illustrated in Figure 13.3. The method is cost effective if sufficient reduction in base course thickness, or subbase improvement, compensates the geogrid cost. Geogrids were used on I-75, Gainesville, FL as shown in Figures 13.4a and 13.4b.

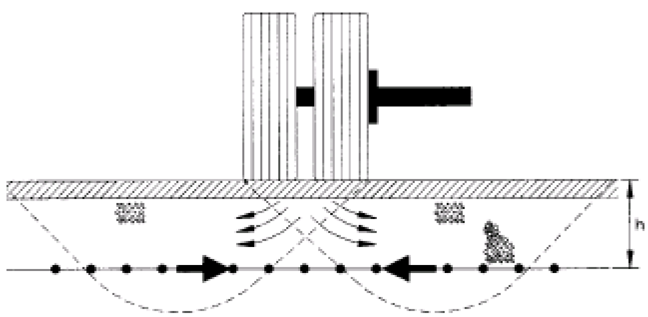


Figure 13.3 Schematic of Tensar Base Course Reinforcement (from Tensar, 1996)

From *Gainesville Sun*, "In the past, the solution would have been to dig out much of the underlying clay layer and then use lime to harden the remaining clay and provide a firm base for road construction. That method worked well in the 1960's, but digging out the clay is expensive. The new solution is plastic – actually three layers of plastic – that are sandwiched around layers of soil and limerock."

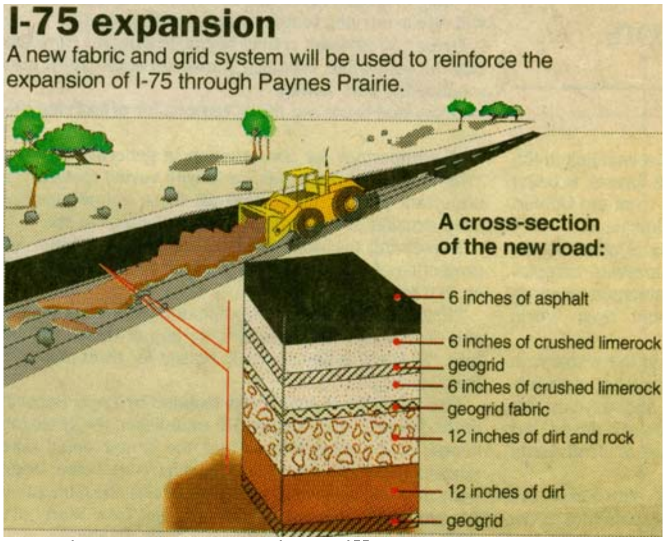


Figure 13.4a ***Gainesville Sun*** **May 6, 1993**

Figure 13.4b Illustrates the reinforcement restraint concept.

Design Procedure

For base reinforcement the methodology is applicable to subgrades with CBR values ≥ 3. The geogrids should be placed at the subgrade-base course interface for base thicknesses ≤ 14”, and at the center of the base course for thicknesses > 14”. For subgrade support of subgrades ≤ 3, the geogrid should be placed at the subgrade-base course interface. The primary benefits of subgrade are: to increase the bearing capacity, and reduce the under-cut and amount of granular fill.

The step–by–step procedure provided by Tensar is as follows:

Spectra Design for Use of Tensar to Reduce Base Course Thickness

DESIGN PROCEDURE (download spectra from www.tensarcorp.com)

The AASHTO design procedure is based upon extrapolations from the AASHO Road Test and experience gained since then. As such, specific designs should be factored into account for local experience. Usually, this results in specification of typical-type pavement structures.

Therefore, a defined pavement structure ~ AC and base thicknesses known ~ will be the assumed starting point for this design procedure. The step-by-step procedure for incorporating a structural geogrid into the design of a defined pavement section is presented below.

Step 1. Define the pavement Geometry

1.a Asphalt Concrete Thickness = _______

1.b Aggregate Base Course Thickness = _______

Step 2. Define the Pavement – Structural

2.a Asphalt Concrete assume a layer coefficient = _______

2.b Aggregate Base Course assume a layer coefficient = _______

2.c Subgrade Soil assume/quantify an average subgrade resilient modulus = _______

2.d Drainage assume/quantify a base course drainage coefficient = _____

Step 3. Compute Pavement Structural Number, SN

$$SN = a_1 D_1 + a_2 D_2 m_2$$

Step 4. Standard Deviation

Assume or quantify a standard deviation, S_o [usually taken as 0.44 or 0.49 for flexible pavements].

Step 5. Reliability

Assume or quantify the reliability level, R [typically 50% to 80% for low volume roads and 80% to 95% for collector and arterial highways]. Determine normal standard deviate, Z_R, for reliability level. (See Table 13.4.)

Step 6. ΔPSI

Assume or select difference between the initial design serviceability index and the design terminal serviceability index [ΔPSI typically taken as 1.7 to 2.2].

Step 7. Estimate the average number of ESALs per year.

Step 8. Estimate the Performance of the Unreinforced Pavement

Total number of ESALs:

the number of ESALs (W_{18}) for the unreinforced pavement structure may be estimated with the following equation:

$$\log_{10}(W_{18}) = Z_R(S_o) + 9.36\log_{10}(SN+1) - 0.20 + \frac{\log_{10}\left[\frac{\Delta PSI}{4.2-1.5}\right]}{0.40 + \frac{1094}{(SN+1)^{5.19}}} + 2.32\log_{10}(M_R) - 8.07$$

Note the equation in Step 8 M_r should be in psi. You can solve the equation using Excel (See Spectra spreadsheet) or nomograph in Appendix A (Figure A13.1).

Table 13.4 Reliability and Standard Normal Deviate Relationship

Reliability, R (percent)	50	60	70	75	80	85	90	91	92
Standard Normal Deviate, Z_R	-0.000	-0.253	-0.524	-0.674	-0.841	-1.037	-1.282	-1.340	-1.405
Reliability, R (percent)	93	94	95	96	97	98	99	99.9	99.99
Standard Normal Deviate, Z_R	-1.476	-1.555	-1.645	-1.751	-1.881	-2.054	-2.327	-3.090	-3.750

The following steps are used for calculating the reduced base course thickness.

Step 8. Select a grade of TENSAR geogrid, either BR1 or BR2 and determine traffic benefit ratio, TBR, value [obtained from Appendix Figures A13.2 – A13-4, or from Tensar]

Step 9. Calculate the reinforced pavement life required due to addition of reinforcement,

$$(W_{18})\alpha = \frac{W_{18}}{1BK}$$

Step 10. Compute the required structural number, SN_{BR} to carry $(W_{18})_R$ using the AASHTO design equation or nomograph.

$$9.36\log_{10}(SN_{BR}+1) + \frac{\log_{10}\left[\frac{\Delta PSI}{4.2-1.5}\right]}{0.40 + \frac{1094}{(SN+1)^{5.19}}} = \log_{10}\frac{W_{18}}{TBR} + 0.20 - 2.32\log_{10}(M_R) - Z_R S_o + 8.07$$

Step 11. Compute the reduced depth of aggregate required for SN_{BR} – using the a_1, D_1, a_2, and m_2 values from the unreinforced case, with

$$D_{2(BR)} = \frac{SN_{BR} - a_2 D_1}{a_2\ m_2}$$

For Step 8, Figures A13.2 – A13.4 are used, but essentially a Traffic Benefit Ratio (TBR) is assumed. Obviously, selection of the TBR governs the amount of reduction in base course thickness.

Pavement thickness reduction is an iterative process using SPECTRA and the reinforced ESALs and reducing the base course thickness until the ESALs match.

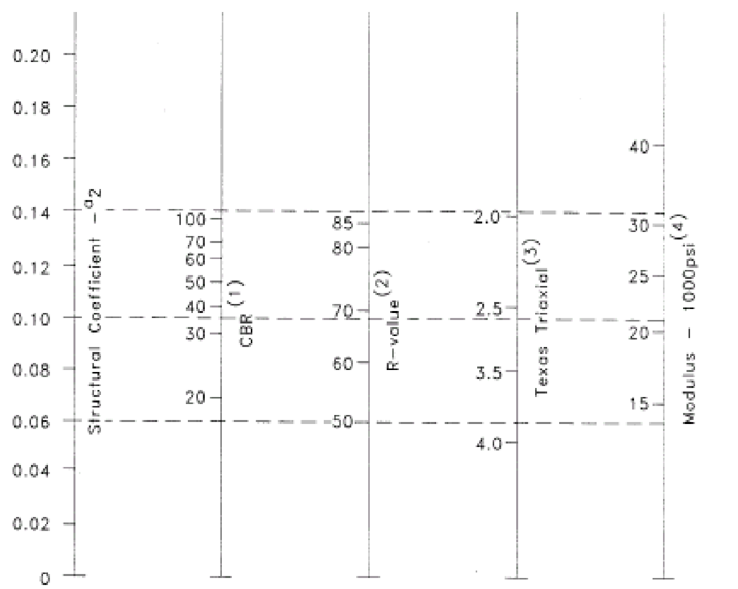
Step 9. This step may appear "goofy" but you need to reduce the ESALs for the reinforced base to plug into the eqn. to solve for W_{18} in Step 10

Using SPECTRA, reduce the thickness of the base course, D_2, until the ESALs for the reduced reinforced D_2 = ESALs for the original unreinforced D_2

Example: Spectra Reinforced Base Course

Granular Base coefficients, a_2, are obtained from Figure 13.5 below:

Again, a defined pavement structure – AC and base thicknesses known – will be the assumed starting point for the design procedure for Option (2) – reduction of aggregate base course thickness. The step-by-step procedure for incorporating a structural geogrid and decreasing the base course thickness, down from a defined pavement section, is presented below.



(1) Scale derived by averaging correlations obtained from Illinois.
(2) Scale derived by averaging correlations obtained from California, New Mexico and Wyoming.
(3) Scale derived by averaging correlations obtained from Texas.
(4) Scale derived on NCHRP Report 128.

Figure 13.5 Variation in Granular Base Layer Coefficient (a_2) with Various Base Strength Parameters (Tensar, 1996)

Table 13.5 Recommended m' Values for Modifying Structural Layer Coefficients of Untreated Base and Subbase Materials (from Table 2.4 AASHTO, 1993 or Tensar, 1996)

Quality of Drainage	Percent of Time Pavement Structure is Exposed to Moisture Levels Approaching Saturation			
	Less Than 1%	**1 – 5%**	**5 – 25%**	**Greater Than 25%**
Excellent	1.40 – 1.35	1.35 – 1.30	1.30 – 1.20	1.20
Good	1.35 – 1.25	1.25 – 1.15	1.15 – 1.00	1.00
Fair	1.25 – 1.15	1.15 – 1.05	1.00 – 0.80	0.80
Poor	1.15 – 1.05	1.05 – 0.80	0.80 – 0.60	0.60
Very Poor	1.05 – 0.95	0.95 – 0.75	0.75 – 0.40	0.40

Step 1. Define the Pavement Geometry

1.a Asphalt Concrete Thickness = 1.5 + 2.5 = 4"

1.b Aggregate Base Course Thickness = 12" .

Step 2. Define the Pavement – Structural

2.a Asphalt Concrete assume a layer coefficient = 0.4 .

2.b Aggregate Base Course assume a layer coefficient = 0.14 . (See Figure 13.6)

2.c Subgrade Soil assume/quantify an average subgrade resilient modulus = 5,000 .

2.d Drainage assume/quantify a base course drainage coefficient = 1.25 . (Table 13.5)

Step 3. Compute Pavement Structural Number, SN = 3.37

$$SN = a_1 D_1 + a_2 D_2 m_2$$

$$= 0.4 (4") + 0.14 (12") (1.25) = 3.70$$

Step 4. Standard Deviation

Assume or quantify a standard deviation, S_o [usually take as 0.44 or 0.49 for flexible pavements].

Step 5. Reliability

Assume or quantify the reliability level R [typically 50% to 80% for low volume roads and 80% to 95% for collector and arterial highways]. (-0.841)

Determine normal standard deviate, Z_R, for reliability level [see Table 13.4].

Step 6. ΔPSI

Assume or select difference between the initial design serviceability index and the design terminal serviceability index [ΔPSI typically taken as 1.7 to 2.2] (use 2.0).

Step 7. Calculate the estimated pavement life, in terms of W_{18} (ESALs), for the unreinforced pavement using the AASHTO design equation or nomograph.

$$\log_{10}(W_{18}) = Z_R(S_o) + 9.36\log_{10}(SN+1) - 0.20 + \frac{\log_{10}\left[\frac{\Delta PSI}{4.2-1.5}\right]}{0.40 + \frac{1094}{(SN+1)^{5.19}}} + 2.32\log_{10}(M_R) - 8.07$$

Using Z_R =-0.841, S_0 = 0.49, SN = 3.70, Δ PSI = 2.0, and M_r = 5,000 psi, results in log (W_{18}) = 6.017841, which gives $10^{6.017841}$ = 1,041,936

Pavement life (or number of years before rehabilitation):

$$\text{Pavement Life} = \frac{W_{18}}{\text{ESALs / year}} = 1{,}041{,}936 \;/\; 500{,}000 = 2.1 \text{ years}$$

Step 8. Select a grade of TENSAR Geogrid, either BR1 or BR2 and determine traffic benefit ratio, TBR, value [obtain from Figures A13.2 – A13.4, or from Tensar] (3/4" rut)

TBR for BR-1 = 3

TBR for BR-2 = 6

Step 9. Calculate the reinforced pavement life required due to addition of reinforcement,

$$(W_{18})_R = \frac{W_{18}}{TBR} = \frac{1{,}041{,}936}{6} = 173{,}656$$

Step 10. Compute the required structural number, SN_{BR}, to carry $(W_{18})_R$ using the AASHTO design equation or nomograph.

$$9.36\log_{10}(SN_{BR}+1) + \frac{\log_{10}\left[\frac{\Delta PSI}{4.2-1.5}\right]}{0.40 + \frac{1094}{(SN+1)^{5.19}}} = \log_{10}\frac{W_{18}}{TBR} + 0.20 - 2.32\log_{10}(M_R) - Z_R S_o + 8.07$$

Using W_{18} = 173,656, results in SN_{BR} = 2.80

Step 11. Compute the reduced depth of aggregate required for SN_{BR} - using the a_1, D_1, a_2, and m_2 values from the unreinforced section, and SN_{BR}, compute $D_{2(BR)}$, the depth of aggregate required for the reinforced case, with

$$D_{2(BR)} = \frac{SN_{BR} - a_1 D_1}{a_2 m_2} = \frac{2.80 - 0.4(4")}{(0.14)(1.25)} = 6.85" \text{ Reinforced}$$

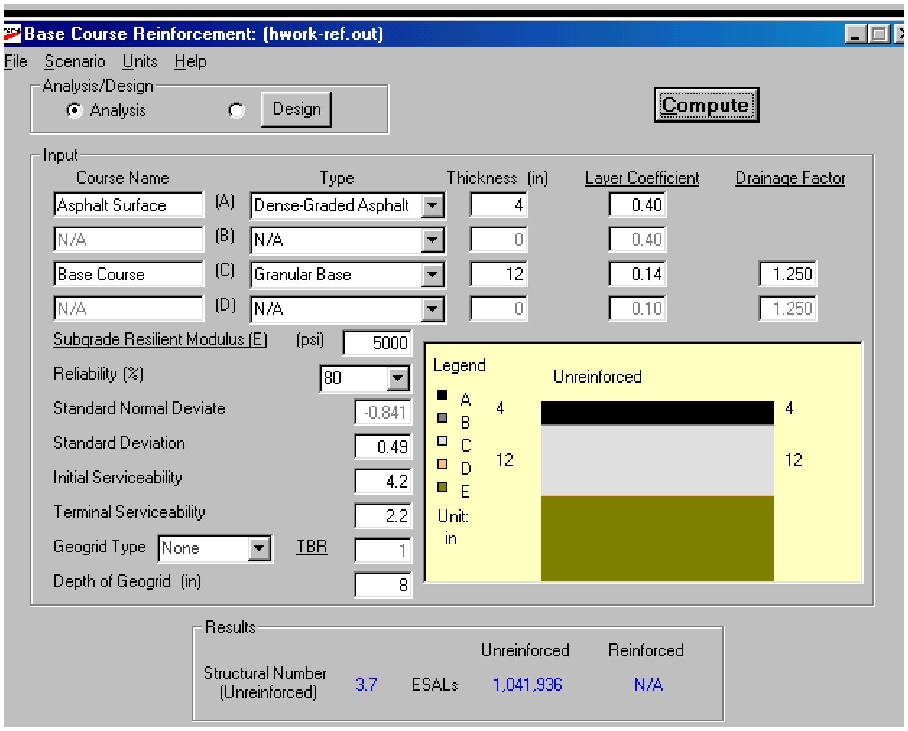


Figure 13.6 Spectra Output Screen for Unreinforced Base Course

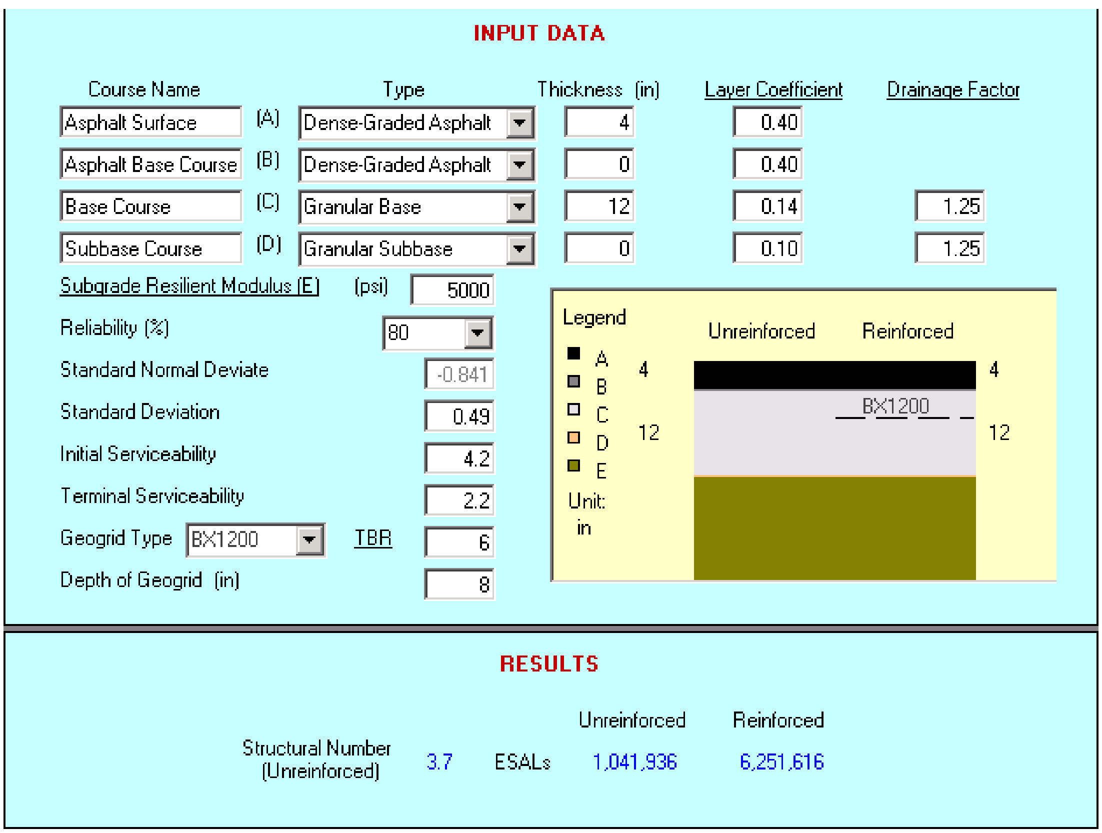


Figure 13.7 Spectra Output Screen for Reinforced Base Course

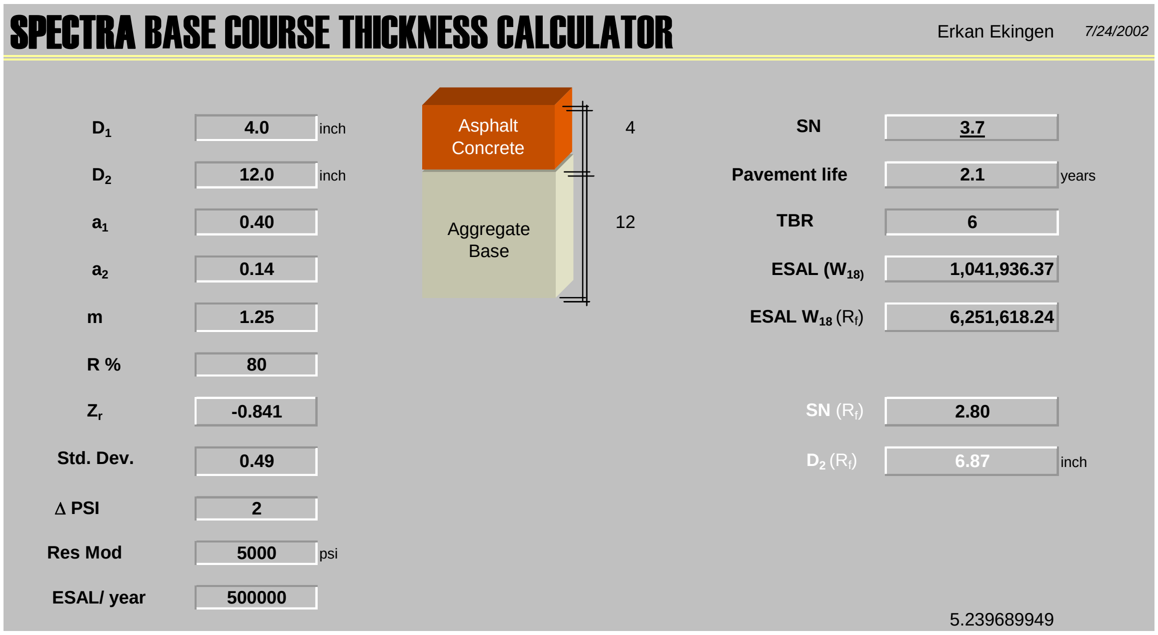


Figure 13.8 UF Spread-sheet for Reinforced Base Course Thickness

DESIGN METHOD FOR TENSAR GEOGRID-REINFORCED UNPAVED ROADS

(Giroud, and Jie Han, 1986 updated by S. Valero and A. Anderson, 17 February 2003 www.tensarcorp.com)

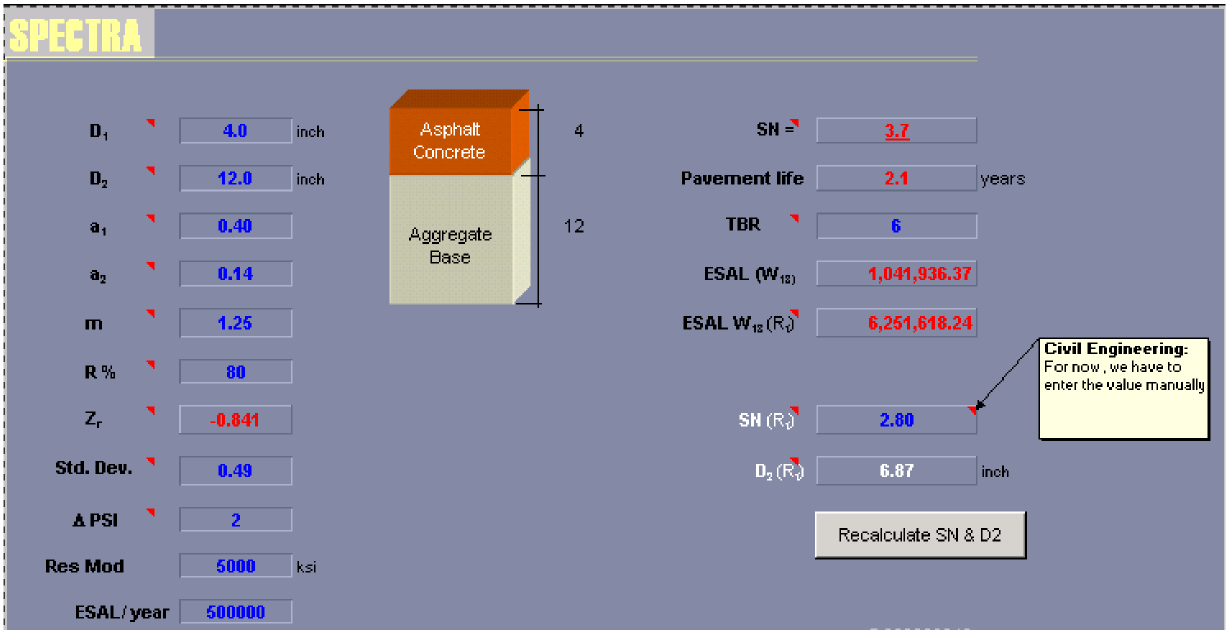
The method employs stress distribution theory to estimate the vertical pressure on the subgrade resulting from a wheel load at the road surface. The aggregate base thickness required to reduce the vertical pressure imparted on the subgrade to a value equal to its estimated bearing capacity is computed. Boussinesq stress distribution theory is used to estimate the maximum vertical stress under the center of a circular loaded area considering the wheel load, tire pressure, and subgrade shear strength in computations. The assumption is that the geosynthetic reinforcement effectively increases the available bearing capacity of the subgrade by changing the failure mode from "local" to "general" bearing capacity failure. In addition, the method also considers the number of load applications and the acceptable rut depth. However, due to a lack of field data the method neglects the obvious contribution of the reinforcement and base course material properties. *Consequently, this method is different than that used for reinforced base courses. Reinforced base courses design depends upon ESAL and TBR, whereas for reinforced subgrade, the tire pressure governs the base course thickness.*

Summary Derivation of the Giroud-Han Method:

The Giroud-Han Method assumes a circular equivalent tire contact area and circular pressure area on the subgrade. The pressure, p, at any depth, h, is then

$$p = \frac{P}{\pi(r + h \times \tan\alpha)^2}$$

Spectra spreadsheet directions: Enter values in "blue," and values in "red" are calculated. For reinforced layers, select appropriate TBR and "recalculate."



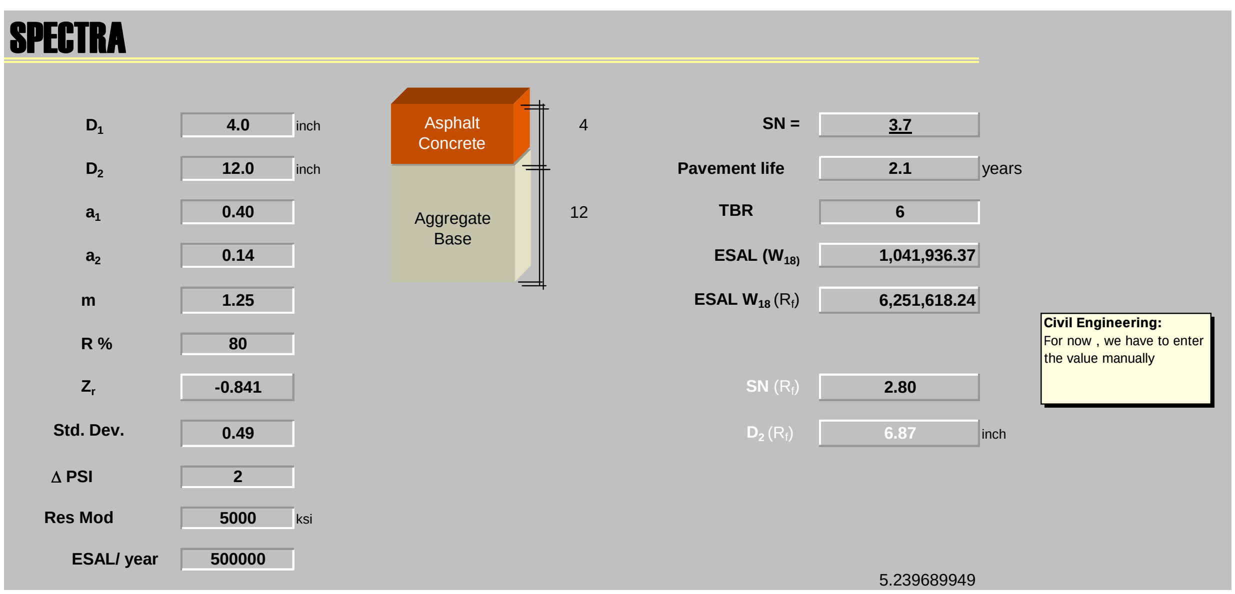


Figure 13.9 UF Spreadsheet Output

where P = wheel load, r = radius of equivalent tire print, α = stress distribution angle. The depth required to distribute the wheel load to a pressure equal to the bearing capacity of the subgrade, q = mc_uN_c is

$$h \geq \frac{r}{\tan\alpha}\left(\sqrt{\frac{P}{\pi r^2 m N_c c_u}} - 1\right)$$

where c_u = undrained shear strength of the subgrade, N_c = bearing capacity factor of the subgrade and m = bearing capacity mobilization coefficient. N_c is taken as 3.14 for unreinforced base courses, 5.14 for geotextile reinforced and 5.71 for Tensar geogrid reinforced base courses. Selection of N_c values is based on classic shallow foundation bearing capacity theory that suggests: 1) N_c = 3.14 for a "local" bearing capacity failure; 2) N_c = 5.14 for a "general" bearing capacity failure where there is a smooth interface (i.e., geotextile) between a footing and soil; 3) N_c = 5.71 for a "general" bearing capacity failure where there is a rough interface (i.e., Tensar geogrid) between a footing and soil. The bearing capacity mobilization coefficient (m) accounts for the fact that only part of the full bearing capacity is developed at any specified rut depth. This coefficient is a function of "r/h" and "s," the rut depth. "m" ranges from nearly 1.0 for relatively thin aggregate bases and a rut depth of 75 mm (3 inches). It is less than 1.0 for thick aggregate bases and/or less than 75-mm rut depths. The function is calibrated using test data.

The stress distribution angle is greater through a high modulus base course than through a lower modulus material. Giroud and Han empirically related the initial stress distribution angle, α_i, through a stiff base over a softer subgrade to the stress distribution angle, α_o, through a homogeneous material and to the ratio of the moduli (or CBR) of the base and subgrade as follows.

$$\tan\alpha_1 = \tan\alpha_o\left[1 + 0.204\left(\frac{E_{bc}}{E_{sg}} - 1\right)\right] = \tan\alpha_o\left[1 + 0.204\left(\frac{3.48\,CBR_{bc}^{0.3}}{CBR_{sg}} - 1\right)\right]$$

where E_{bc} and E_{sg} = the modulus of the base course and subgrade, respectively and CBR_{bc} and CBR_{sg} = the base course and subgrade CBR, respectively. Data from North Carolina State University (Gabr, 2001) was used to correlate the stress distribution angle with the number of load cycles, N, yielding:

$$\frac{1}{\tan\alpha} = \frac{1 + k\,\log N}{\tan\alpha_1}$$

where: k is an empirically derived constant depending on the base course thickness and geogrid reinforcement property J, the aperture stability modulus.

The bearing capacity mobilization coefficient was calibrated and the equations combined to arrive at the following design equation for the minimum required thickness of the base course:

$$h = \frac{0.868 + \left(0.661 - 1.006J^2\right)\left(\frac{r}{h}\right)^{1.5}\log N}{1 + 0.204\left[R_E - 1\right]}\left[\sqrt{\frac{\frac{P}{\pi r^2}}{\left(\frac{s}{f_S}\right)\left[1 - 0.9\exp\left(-\left(\frac{r}{h}\right)^2\right)\right]N_c\, f_C\, CBR_{sg}}} - 1\right]r$$

where: h = required base course thickness (m); J = geogrid aperture stability modulus (m-N/°); N = number of axle passages; P = wheel load (kN); r = radius of the equivalent tire contact area (m); $R_E = 3.48(CBR_{bc})^{0.3}/CBR_{sg}$ = limited modulus ratio of base course to subgrade soil, maximum value = 5.0, depth (mm); f_s = factor equal to 75 mm; N_c = bearing capacity factor; $c_u = f_c\ CBR_{sg}$ and f_c = factor equal to 30 kPa; and CBR_{sg} = CBR of the subgrade soil. And N_c = 3.14 and J = 0 for unreinforced base course,

N_c = 5.14 and J = 0 for geotextile reinforced base course,

N_c = 5.71 and J = 0.32 m-N/deg for Tensar BX1100 reinforced base course, and

N_c = 5.71 and J = 0.65 m-N/deg for Tensar BX1200 reinforced base course.

In application of the Giroud-Han method, it is currently suggested, based on a field study of unreinforced bases, that the modulus ratio $R_E = E_{bc}/E_{sg}$ be limited to a maximum value of 5.0 to account for the inability to effectively compact base course material over very soft subgrades. Consequently the term $3.48\ (CBR_{br})^{0.3}/CBR_{sg} \leq 5.0$. Since the required base course thickness, h, appears on both the LHS and RHS of the equation, an iterative solution is required. The application of the method is easily applied using Tensar's Spectra 2 code.

Example of Subgrade Reinforcement

Find the unreinforced vs. reinforced sub-base thickness for an 18 kip (80 kN) wheel load with 80 psi tire pressure overlying a subgrade with CBR = 3.3. Assume the subbase has a CBR = 20 and the rut depth is limited to 1.5 inches (38.1 mm) for 500,000 passes. The following solution from Spectra reveals a thickness reduction of 8.7 or 14.8 inches for BX 1100 or BX 1200, respectively.

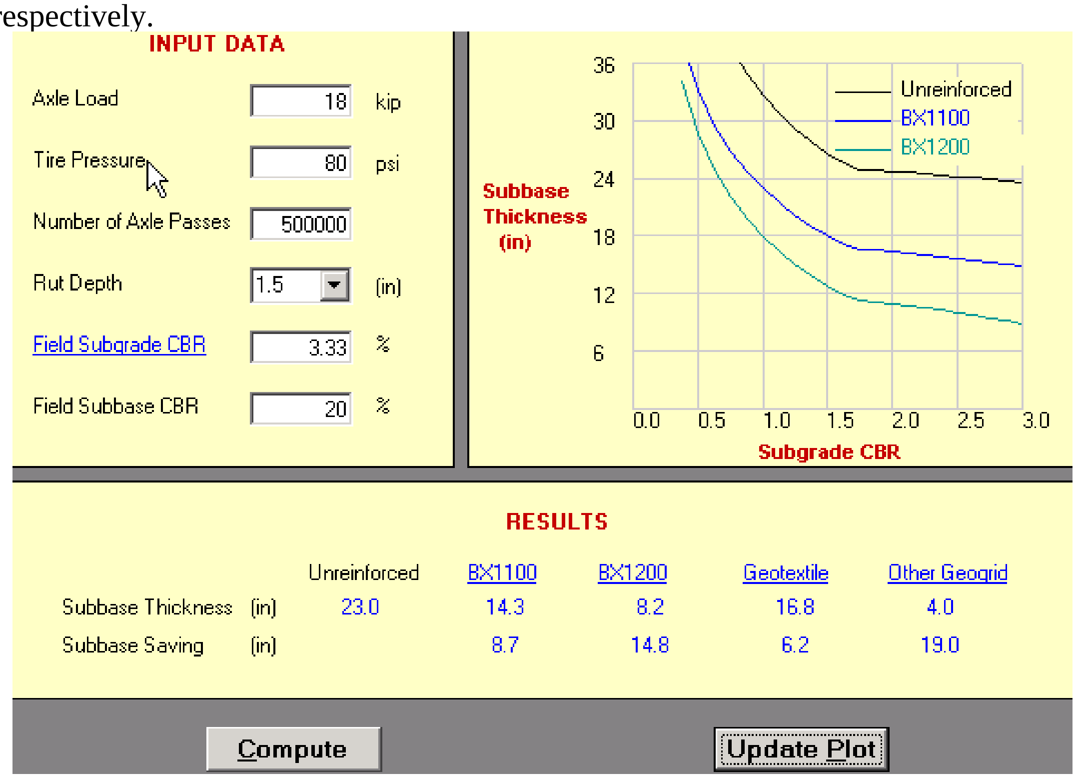


Figure 13.10 Output Screen for Spectra Reinforced Subgrade

Calculation verification

a. $$h = \frac{0.868 + \left(0.661 - 1.006J^2\right)\left(\frac{r}{h}\right)^{1.5} \log N}{1 + 0.204\left[R_E - 1\right]} \left[\sqrt{\frac{\frac{P}{\pi r^2}}{\left(\frac{s}{f_S}\right)\left[1 - 0.9\exp\left(-\left(\frac{r}{h}\right)^2\right)\right] N_c\, f_C\, CBR_{sg}}} - 1 \right] r$$

b. For unreinforced subbase, J = 0

c. Radius, r = $r = \sqrt{P/\pi p} = \sqrt{18{,}000\text{lbs}/\pi(80\text{psi})} = 8.5'' = 0.215\text{m}$

d. R_E = $3.48\,(CBR_{bc})^{0.3}/CBR_{sg}$ = $3.48\,(20)^{0.3}/3.33$ = 2.567

e. Rut depth, s = 1.5” = 38.1mm; f_s = 75 mm (default)

f. N_c nonreinforced = 3.14, and CBR = 3.33 → C_u (kPa) = $c_u = f_C\ CBR_{sg}$ = 30 x 3.33 = 100 kPa

g.
$$h = \frac{0.868 + (0.661 - 1.006(0)^2\ )(.215/h)^{1.5} \log 500000}{1 + 0.204\left[2.567 - 1\right]} \left[\frac{80/0.215^2\pi}{\left(\frac{38.1\text{mm}}{75\text{mm}}\right)\left[1 - 0.9\exp(-(.215/h)^2\right]3.14 \times 100\text{kPa}} \right]^{0.5} - 1\ ](.215\text{m})$$

by iteration for h; h = 0.82m vs. 23” (0.58m) Spectra

by increasing C_u = 167kPa, h iterates to 0.58m. Consequently, selection of C_u correlation is paramount for agreement.

REFERENCES

AASHTO (1986) Guide for Pavement Structures, AASHTO.

Barber, E.S. and Sawyer, C.L. (1952), Highway Subdrainage, Proc. Highway Res. Board, pp. 643-666

Casagrande, A. and Shannon, W.L. (1952) Base Course Drainage for Airport Pavements, Trans. ASCE pp. 792-814

Cedergren, H.R. (1973) Development of Guidelines for the Design of Subsurface Drainage Systems for Highway Pavement Structural Sections, FHWA-RD-73-14, Federal Highway Administration.

Gabr, M. (2001). Cyclic Plate Loading Tests on Geogrid Reinforced Roads. Research Report to Tensar Earth Technologies, Inc., NC State University, 43 p.

Giroud, J.P., 2000, "Filter Criteria," in Jubilee Volume 75th Anniversary of K. Terzaghi's Erdbaumechanik (Soil Mechanics), Technical University, Vienna, Austria, Vol 5/2000, Brandl, H., editor. (from Landfilldesign.com).

Huang, Y.H. (1993) Pavement Analysis and Design, Prentice-Hall Pub.

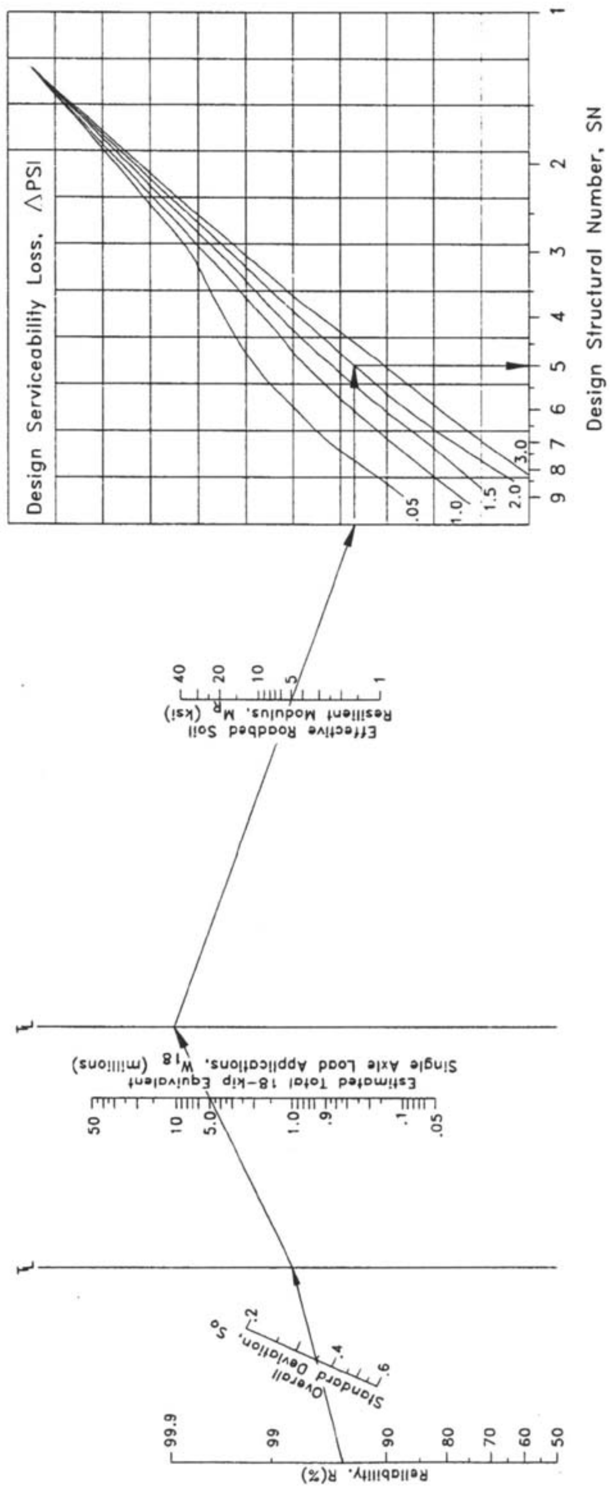
Landfilldesign.com (2003) Geotextile Filter Design Calculator.

Moulton, L.K. (1980) Highway Subdrainage Design, Rept. FHWA-TS-80-224, Federal Highway Administration.

Tensar (1996) Design Guideline for Flexxible Pavements with Tensar Geogrid Reinforced Base Layers, Tech. Note TTN:BR96, www.tensarcorp.com

Tensar (1998) A Review of Geosynthetic Functions and Applications in Paved and Unpaved Roads TTN BR 11.

Appendix to Chapter 13



$$\log_{10}(W_{18}) = Z_R(S_o) + 9.36\log_{10}(SN+1) - 0.20 + \frac{\log_{10}\left[\frac{\Delta PSI}{4.2-1.5}\right]}{0.40 + \frac{1094}{(SN+1)^{5.19}}} + 2.32\log_{10}(M_R) - 8.07$$

Figure A13.1 Design Chart for Flexible Pavements (AASHTO, 1993)

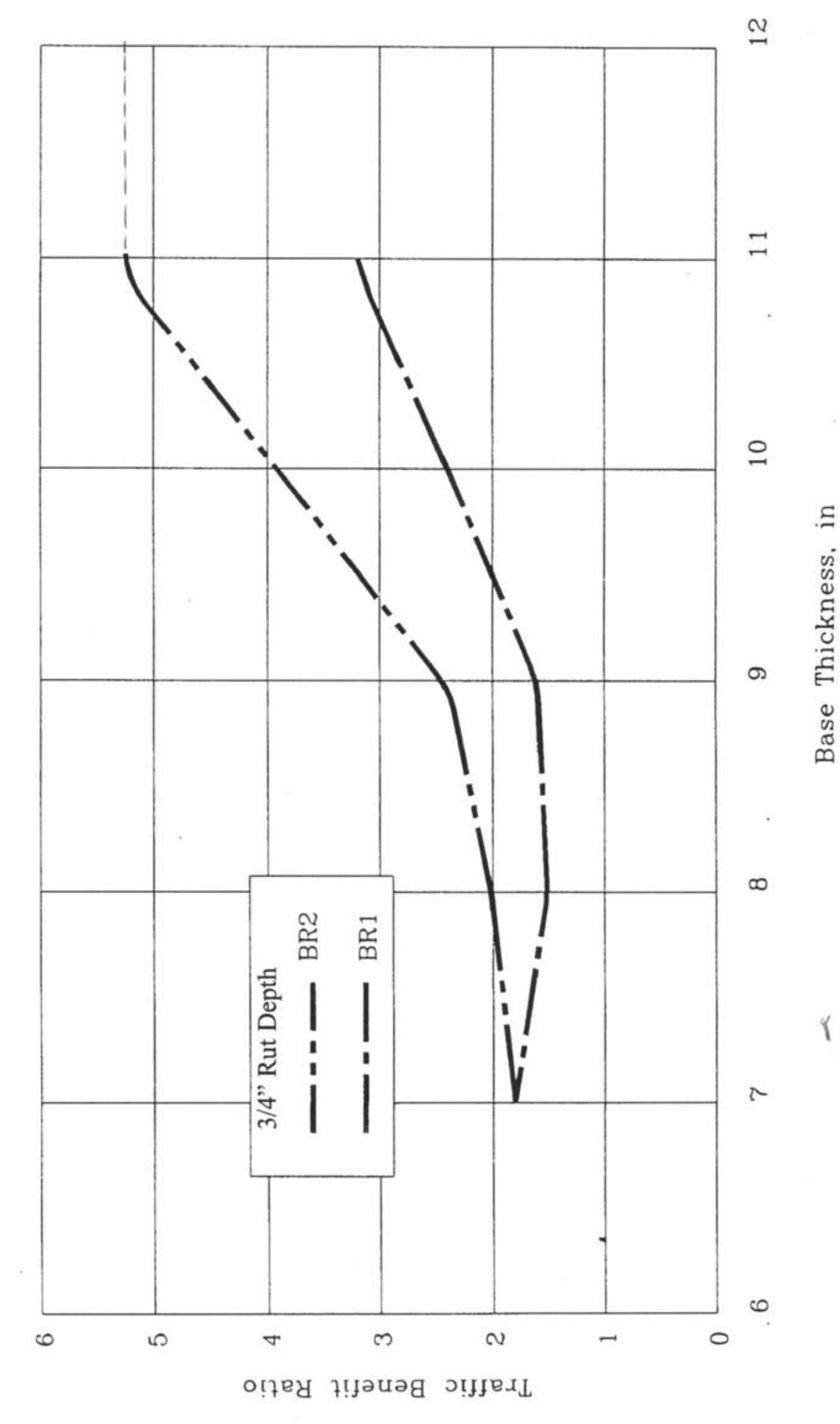


Figure A13.2 Traffic Benefit Ratio versus Base Course Thickness for ¾ inch Rut Depth

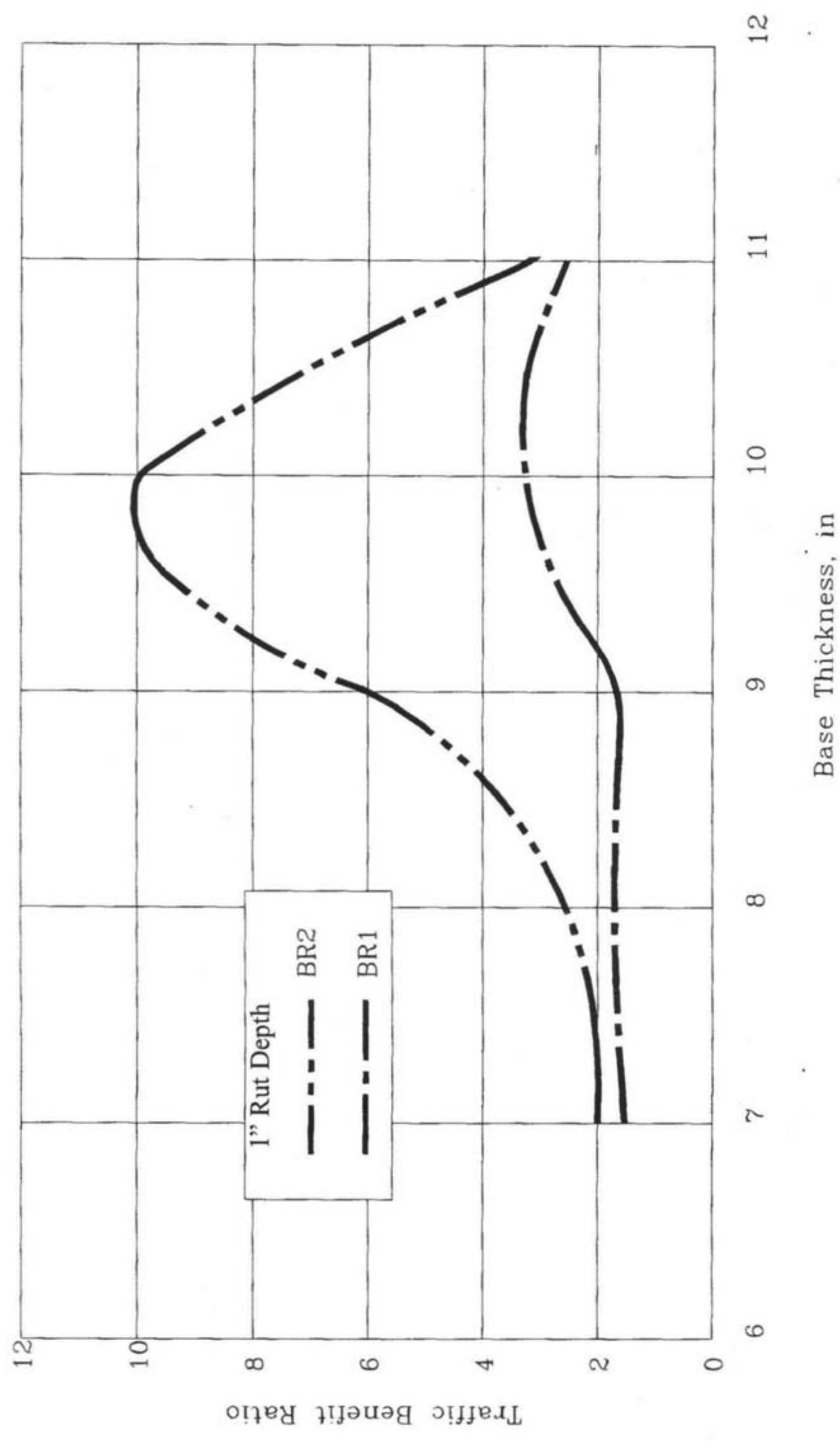


Figure A13.3 Traffic Benefit Ratio versus Base Course Thickness for 1 inch Rut Depth

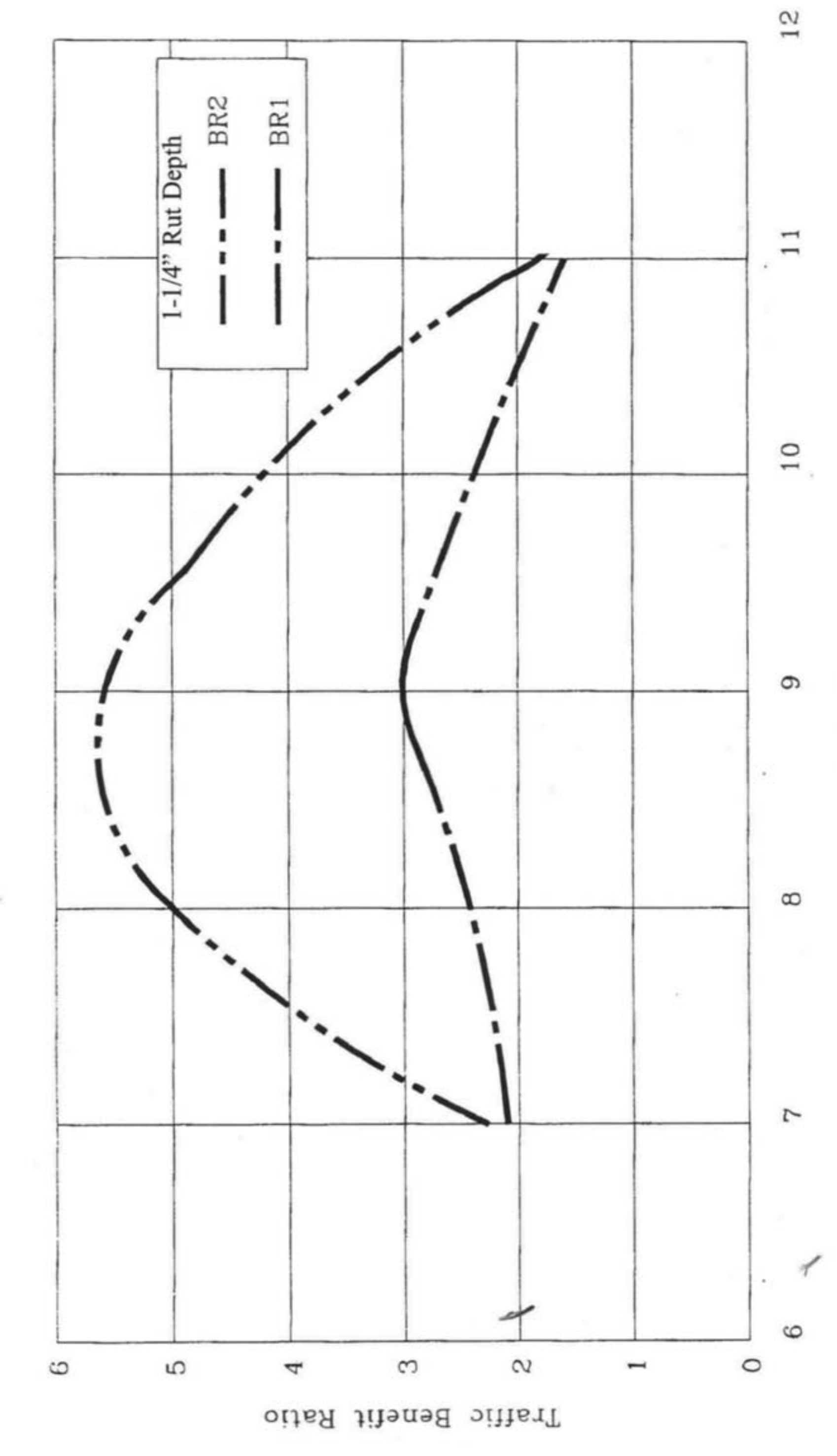


Figure A13.4 Traffic Benefit Ratio versus Base Course Thickness for 1 ¼ inch Rut Depth

Chapter 14

LIGHTWEIGHT FILLS

Exactly as the name implies, this ground modification technique involves the replacement of geomaterials with materials with a lower unit weight. Lightweight fills are often used when the magnitude of effective stress needs to be reduced. Typical applications include, reduction of settlement of compressible materials, reduction of driving moments in slope stability, and reduction of active force behind retaining walls. Figures 14.1, 14.2, and 14.3 show common applications of lightweight fills.

Figure 14.1 Geofoam Blocks Placed as Fill

Figure 14.2 Tire Shreds

Figure 14.3 Fly Ash as Embankment Fill

Application:

1) Lower γ, reduction of vertical stress. Reduce driving forces for bearing capacity, slope stability, and settlement.
2) Better "geotechnical properties" such as c or ϕ that is favorable for increasing stability of embankment against a slope stability type failure.
3) Have compressibility similar to natural soil thus pavements can be built either directly upon these fills or with a buffer of soil between.(shredded tires and wood fibers are much greater- overbuild)
4) Some LWFs have a low Poisson's ratio, ν, when combined with lower γ is favorable for reducing lateral stresses that would be transmitted to retaining walls or tunnels.
5) Granular LWFs are relatively pervious and drain rapidly which is favorable for sub-grade support.
6) Some LWFs can be placed in wet or cold weather that would restrict or preclude conventional earthwork operations.
7) The low γ lightweight fill materials are beneficial in seismic areas because the seismic internal force is directly related to fill density

Limitations:

1) Availability of materials. Wood fiber in lumber areas, fly ash in industrial areas etc. Transportation costs are a factor.
2) Construction methods. Specialized methods – specialized equipment. Cost.
3) Durability of fill deposits. Geofoams sensitive to petroleum, wood fiber can decompose.
4) Environmental concerns. Leachates from tire shreds etc.
5) Geothermal properties. LWFs geothermal properties different than soil. Can lead to accelerated deterioration of pavements.

Table 14.1 Properties of Lightweight Fill Materials

Fill Type	Range in Density kg/m3	Range in Specific Gravity	Approxmiate Cost $/m3
Geofoam (EPS)	12 to 32	0.01 to 0.03	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]
Fly Ash	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]
Soil	1500 to 2000	2.60 to 2.70	variable

[1] Price includes transportation cost

[2] FOB plant

[3] Mixed at job site using pumps to inject foaming agents into concrete grout mix

The following Tables 14.2 through 14.8 are FHWA (Elias, 1998) guidelines for the use of many common lightweight fill materials.

ADDITIONAL NOTES ON TIRE SHREDS

Because of past interest by FDOT in the use of tire shreds, some additional guidelines from Humphrey 1998 are included as follows:

- Tire shreds used for: (1) light-weight fills, (2) Drainage, (3) Low earth pressure against abutments, (4) Thermal insulation, (5) Land fill drains and cover.
- Disposal Problem: 850 million tires + 253 million new/ yr. 75 tires / cu. yd of fill.
- Tire shreds are typically 2.5 to 3.0 inches.
- Light-weight fill. Typical unit wt. 40 -60 pcf, high shear resistance to slope stability, reduced wt. for compressibility of underlying soft clays.

Design with Shredded Tires as Lightweight Fills

Since tires are compressible, we need to calculate the amount of overbuild.

1. Estimate initial uncompressed dry unit wt, γ. (for 3-in shreds use 40 pcf)
2. Estimate initial water content, w, as $\gamma_m = \gamma_d\,(1+w)$; use w = 3-4%
3. Determine vertical stress in center of tire shred layer,
   $\sigma_{v-center} = t_{overburdensoil}\gamma_{soil} + (t_{shreds}/2)(\gamma_{shreds})$, thickness of shred layer ÷2 for mid-depth.

Table 14.2 Wood Fiber Design and Construction Guidelines (FHWA, 1998)

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:	1×10^{-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 14.3 Air Cooled Blast Furnace Slag Design and Consideration Guidelines

(FHWA, 1998)

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 14.4 Fly Ash Design and Construction Guidelines (FHWA, 1998)

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 1×10^{-6} to 1×10^{-9} m/s
Compressibility:	Cc = 0.05 to 0.37 Ccr = 0.006 to 0.04
Grain Size Range:	.005 to 0.74 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.	
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 fly ash. 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 14.5 Boiler Slag Design and Construction Guidelines (FHWA, 1998)

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:	0.5 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 14.6 Expanded Shales and Clays Design and Construction Guidelines (FHWA, 1998)

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 36 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 14.7 Shredded Tires Design and Construction Guidelines (FHWA, 1998)

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 35 mm/sec
Gradation:	100 to 200 mm
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 14.7 Continued

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 14.8 Expanded Polystyrene (EPS) Design and Construction Guidelines

(FHWA, 1998)

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.

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.

Construction Considerations:

The subsoil should be leveled before placement of geofoam blocks. A layer of san/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.

4. Estimate percent compression, ε_v using $\sigma_{v\text{-center}}$ (for $\sigma_{v\text{-center}}$ = 5 psi, ε_v = 20%)
5. Determine compressed moist unit wt. $\gamma = \gamma_i /(1-\varepsilon_v)$

Design Details:

1. Large deformations may be required for shreds to develop shear strength. But stability not a problem for "reasonable" side slopes.
2. Shreds are wrapped by geotextile to prevent inflow of soil into shreds.
3. Don't mix shreds and soil: (a) difficult to mix (b) additional construction costs, (c) poor mixing leads to increased settlement, (4) the more you mix soil, the more you loose the beneficial properties of shreds.
4. Environmentally fills above gwt - All metals below drinking water standards, but metals with secondary standards Mg and Fe exceeded - from steel belts. For below gwt - Drinking water standards OK, but Mg and Fe exceeded. Conclusion: Use surface soil cover and drainage ditch.
5. Exothermic reactions: thickness < 5 ft OK, thickness < 10-18ft OK, t > 20 ft Problems. therefore separate layers with soil and keep 3m (15ft) max.

Construction:

1. Cu. yd. vs. ton – yd. cu. loose or compacted, therefore better to purchase by ton.
2. Supply of shreds may need to stock-pile.
3. Spread shreds with track mounted dozer, tires get flats.
4. Compact using vibratory smooth wheeled roller or vib. sheepsfoot. 12 in lifts 6-8 passes.
5. The “spring” in tires makes difficult to compact overburden soils. For granular soils, it is better to compact wet of optimum.

PERTINENT WEB LINKS FOR REVIEW

Geo Foam

http://www.geofoam.org/

http://geofoam.syr.edu/

http://www.geosynthetic-institute.org/Products%5Cgf.htm

http://www.thermafoam.com/geofoam.html

http://www.polyfoam.com/geo.html

Tire Shreds

http://www.dot.state.ny.us/tech_serv/geo/tires/tire_pix.html

http://www.umaine.edu/research/UMTRoadAgain.htm

Blast Furnace Slag

http://www.tfhrc.gov/hnr20/recycle/waste/

http://www.edwclevy.com/materialguides/BaseMarket/3X1BlastFurnaceSlag.htm

Fly Ash

http://www.tfhrc.gov/hnr20/recycle/waste/

http://www.ukqaa.org.uk/BGuide02/BPGuide2Sept2001.htm

Boiler Slag

http://www.tfhrc.gov/hnr20/recycle/waste/

Expanded Shale and Clay

http://www.hpbhaydite.com/haydite_geotech1.htm

Foamed Concrete

http://www.alliedfoamtech.com/Appconc.htm

REFERENCES:

Elias, V., Welsh, J., Warren, J., and Lukas, R., "Ground Modification Technical Summaries," Federal Highway Administration Publication No. FHWA-SA-98-086, September 1998.

Humphrey, Dana, (1998) Civil Engineering Applications of Tire Shreds-Short Course for FDOT, FDEP, Gainesville, FL.