Chapter 19

GROUND IMPROVEMENT

Final

SCDOT GEOTECHNICAL DESIGN MANUAL

June 2010

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CHAPTER 19

GROUND IMPROVEMENT

19.1 INTRODUCTION

According to Ground Improvement Methods, Volume I, FHWA NHI-06-019, August 2006, "One of the major functions of geotechnical engineering is to design, implement and evaluate ground improvement schemes for infrastructure projects. During the last 25 years significant new technologies and methods have been developed and implemented to assist the geotechnical specialist in providing cost-effective solutions for construction on marginal or difficult sites." The ground improvement methods discussed in this Chapter are based on the contents of Ground Improvement Methods, Volumes I and II but should not be the complete discussion of ground improvement methods. For simplicity Ground Improvement Methods, Volumes I and II, FHWA NHI-06-019 and FHWA NHI-06-020, August 2006 will be referenced as Ground Improvement Methods. The engineer is should consultant each volume as required for more details concerning a specific ground improvement method. In keeping with the geotechnical philosophy described in Chapter 7, it is incumbent on the geotechnical engineer/professional to be aware of new and innovative ground improvement ideas. If a new or innovative ground improvement method is to be used on an SCDOT project, approval must be first obtained from the GDS and the PCS/GDS. The approval process will consist of a minimum of engineering design, the desired outcome, construction methodology, and availability of construction experience/contractors to perform the specified type of work.

Ground improvement construction methods are used to improve poor/unsuitable subsurface soils and/or to improve the performance of embankments or structures. These methods are used when replacement of the in-situ soils is impractical because of physical limitations, environmental concerns, or is too costly. Ground improvement methodologies have the primary functions to:

- Increase bearing capacity, shear, or frictional strength,
- Increase density,
- Control deformations,
- Accelerate consolidation,
- Decrease imposed loads,
- Provide/increase lateral stability,
- Form seepage cutoffs or fill voids,
- Increase resistance to liquefaction, and
- Transfer embankment and/or ERS loads to more competent layers.

According to Ground Improvement Methods, "There are three strategies available to accomplish the above functions representing different approaches."

1. Increase shear strength, density, and/or decrease compressibility of foundation soil,
2. Use lightweight fills to significantly reduce the applied load on the foundation soil, and
3. Transfer the load to a more competent (deeper) foundation soil.

Ground Improvement Methods recommends a sequential design process that includes a sequence of evaluations that proceed from simple to more detailed. This process identifies the best method and is defined in Table 19-1.

Table 19-1, Ground Improvement Design Process (modified from Ground Improvement Methods – August 2006)

Step	Process
1	Identify potential poor ground conditions, including extent and type of negative impact
2	Identify or establish performance requirements (see Chapter 10)
3	Identify and assess any space or environmental constraints
4	Assessment of subsurface conditions – type, depth and extent of poor soil as well as groundwater table depth and assessment of shear strength and compressibility
5	Preliminary selection – takes into account performance criteria, limitations imposed by subsurface conditions, schedule and environmental constraints, and amount of improvement required (Table 19-2 should be used in this selection process)
6	Preliminary design
7	Comparison and selection – selection is based on performance, constructability, cost, and any other relevant project factors

Table 19-2, Ground Improvement Categories, Functions and Methods (modified from Ground Improvement Methods – August 2006)

Category	Function	Method
Consolidation	Accelerate consolidation and increase shear strength	1.) Prefabricated Vertical Drains 2.) Surcharge
Load Reduction	Reduce load on foundation and reduce settlement	1.) Geofoam 2.) Foamed Concrete 3.) Lightweight fill
Densification	Increase density, bearing capacity, and frictional strength of granular soils. Decrease settlement and increase resistance to liquefaction	1.) Vibro-Compaction 2.) Dynamic Compaction by falling weight impact
Reinforcement	In soft foundation soils, increases shear strength, resistance to liquefaction, and decreases compressibility	1.) Stone Columns
Deep Soil Mixing	Physio-chemical alteration of foundation soils to increase their tensile, compressive, and shear strength; to decrease settlement; and/or provide lateral stability and/or confinement	1.) Wet mixing methods 2.) Dry mixing methods
Grouting	To form fill voids, increase density, increase tensile, and compressive strength	1.) Permeation Grouting 2.) Compaction Grouting 3.) Jet Grouting
Load Transfer	Transfer load to deeper bearing layer	1.) Column Supported Embankment (CSE)

As indicated in Step 7, the cost of the ground improvement method must be considered in the selection process. Contact the PCS/GDS for cost information for ground improvements methods previously used by SCDOT. For ground improvement methods not previously used, every effort should be made to contact at least three contractors to obtain approximate pricing information.

According to Ground Improvement Methods, "The success of any ground improvement method is predicated on the implementation of a QA/QC program to verify that the desired foundation improvement level has been reached. These programs incorporate a combination of construction observations, in-situ testing and laboratory testing to evaluate the treated soil in the field. Details are provided in the *following Sections*." Italics in quotations indicate words added or changed to correspond to this Manual. The costs for the QA/QC program need to be added to the total cost of the ground improvement method.

19.2 PREFABRICATED VERTICAL DRAINS

Prefabricated vertical drains (PVDs), also commonly called wick drains, are used to accelerate consolidation of compressible cohesive soils to speed settlement and strength gain. The use of the term wick drains is a misnomer since water is not wicked out of the ground by the drains under capillary tension, but rather water flows from compressible clays under a pressure gradient induced by excess pore pressures associated with the placement of permanent fill and/or surcharge fill (see Figure 19-1).

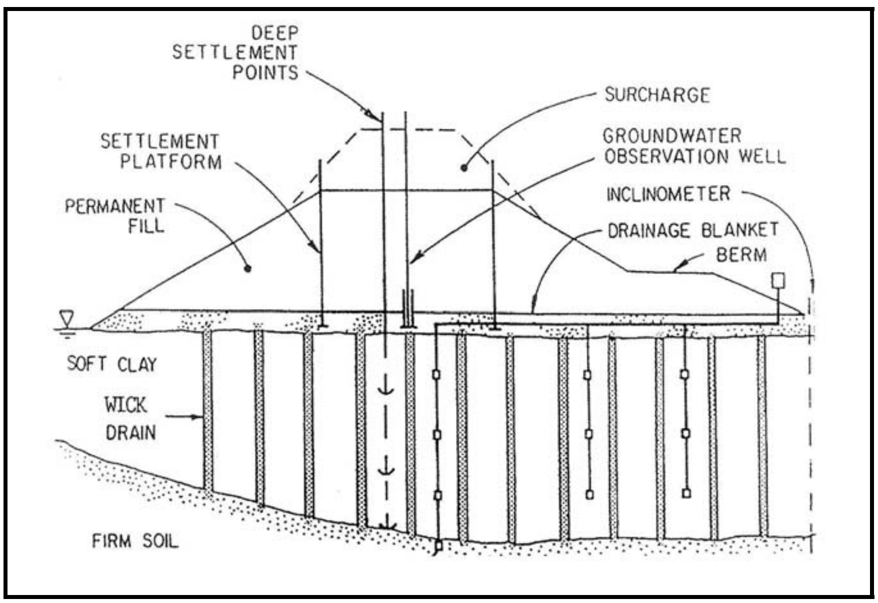


Figure 19-1, PVD Installation for a Highway Embankment (Ground Improvement Methods – August 2006)

PVDs have numerous advantages some of which include economy, installation speed, continuity of drain, and minimal displacement. Additional advantages are presented in Ground Improvement Methods, which should be consulted for greater details on this method. There are also some disadvantages to the PVDs which include greater quantities, no compressive strength, headroom limitations, and material must be properly handled and stored. It is noted that these disadvantages are in relation to the use of sand drains. The subsurface soils must be evaluated to determine the feasibility of using PVDs. The evaluation factors are provided below:

- Moderate to high compressibility
- Low permeability
- Full saturation
- Final embankment loads must exceed maximum preconsolidation stress (σ'_p or p'_c)
- Secondary compression must not be a major concern
- Low-to-moderate shear strength
- Soils normally to slightly overconsolidated (OCR < 1.5)

PVDs are thin plastic strips (about 1/8 inch thick by 4 inches wide) consisting of a rigid core sheathed in filter fabric. They have generally replaced sand drains and the design theories were adapted from sand drain design. To accelerate the rate of settlement, PVDs, are typically installed on a regular grid pattern, either triangular or rectangular, to reduce the flow distance for dissipation of excess pore water pressures associated with the placement of fill. Stone columns discussed later in this Chapter also can provide vertical drainage and similar methods can be applied to evaluate their effect on settlement rates.

19.2.1 <u>Design Concepts</u>

The primary purpose of PVDs is to reduce the length of the drainage path, thereby decreasing the time for settlement and strength gain to occur. Prior to selecting the use of PVDs, predictions of the amount and rate of settlement (see Chapter 17) both during and after construction are required. The amount of settlement must meet the performance criteria provided in Chapter 10. In addition, the stability of the embankment during the placement of the fill materials should also be ascertained. If the stability becomes questionable during construction, then vertical staging may be required. Chapter 17 discusses the stability of the embankment and vertical staging if required. Field testing (SPT, CPT and/or DMT) is required to determine if pre-drilling is necessary. The principle of PVD design is the selection of the type, spacing and length of the drains to accomplish the required performance limit (degree of consolidation) within a specified time.

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

$$\boldsymbol{t = \frac{D^2}{8C_h}\left(F(n) + F_S\right)\ln\left(\frac{1}{1-\overline{U_h}}\right)} \qquad \text{Equation 19-1}$$

Where,

t = Time required to achieve desired average degree of consolidation

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

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

C_h = Consolidation Coefficient for horizontal drainage

F(n) = Drain spacing factor (see Equation 19-2)

d = Equivalent circular drain diameter
F_S = Factor for soil disturbance

This equation does not include any consolidation due to vertical drainage. It is noted that the predicted settlement amounts and rates (discussed in Chapter 17) are based on vertical drainage.

$$\boldsymbol{F(n) = \ln\left(\frac{D}{d}\right) - 0.75} \qquad \text{Equation 19-2}$$

The following sections contain a discussion of each of these components.

19.2.1.1 Determination of F_S

Soil disturbance is typically ignored except for highly plastic (PI > 21), sensitive (S_t > 5) soils, where the Consolidation Coefficient for vertical drainage (C_v) has been accurately determined. For these soils an $F_S \approx 2$ should be used, otherwise use $F_S = 0$. Sample disturbance is more pronounced at drain spacings of less than 5 feet or by the use of large, thick anchor plates.

19.2.1.2 Determination of C_h

The horizontal Consolidation Coefficient (C_h) can be obtained only through laboratory consolidation testing of high quality samples. Even with high quality samples and testing, the results of the testing can be off by 50 percent of the actual values. Normally C_h is greater than C_v. A conservative approach is set C_h equal to C_v, without direct measurements of C_h. However, for design, C_h can be taken as 1.2 to 1.5 C_v, if no or only slight evidence of layering is evident in partially dried clay samples. If layering of silt and sand in discontinuous lenses is evident, C_h may be 2 to 4 C_v. The horizontal Consolidation Coefficient may be assessed in the field using CPT instrumentation and allowing for pore pressure dissipation.

19.2.1.3 Determination of d

The equivalent circular drain diameter (d) of a PVD has been determined using various methods. Diameters ranging from 1.6 to 5.5 inches have been used for the equivalent circular drain diameter, with the most common being 2.4 inches.

19.2.1.4 Determination of $\overline{U_h}$

The average degree of consolidation ($\overline{U_h}$) should meet the performance limit requirements of Chapter 10. Vertical consolidation can contribute significantly to the total amount of vertical movement and should be considered in the development of the degree of consolidation required.

19.2.1.5 Determination of D

According to Ground Improvement Methods, "When using an equilateral triangular pattern, *the diameter of the cylinder of influence (D)*, is 1.05 times the spacing between each drain. In a square pattern, D is 1.13 times the spacing between drains. Typically, to achieve approximately

90 percent consolidation in 3 to 4 months, designers often choose drain spacing between 3 to 5 feet in homogeneous clays, 4 to 6 feet in silty clays and 5 to 6-1/2 feet in coarser soils."

19.2.1.6 Determination of t

The time (t) is the duration required to achieve the desired average degree of consolidation ($\overline{U_h}$) for a diameter of the cylinder influence (D) and drain diameter (d). According to Ground Improvement Methods, "There are three basic variables that can be manipulated in order to achieve a desired result from Equation *19-1*. These variables are time, PVD spacing, and surcharge. *In order to increase the PVD spacing and reduce the number of PVDs installed, the surcharge can be increased to provide the same amount of consolidation over the same time period*." The addition of surcharge and keeping the PVD spacing the same has the effect of reducing the time for consolidation to occur. Typically, time is used as a constant (normally set to meet a specific construction schedule) and the amount of surcharge and the PVD spacing are used as variables.

19.2.1.7 Computer Software

Simple applications can be analyzed with hand calculations or with the use of a spreadsheet program to facilitate sensitivity studies. The computer program, FoSSA 1.0, can be used for analyses where the rate of loading becomes more complex and hand solutions become impractical.

A complete set of the design calculations prepared in accordance with this Chapter shall be provided. The determination of surcharge amounts and PVD spacing shall be fully documented with all design calculations. Submitted calculations (including computer input and output) shall include all assumptions used in the analysis. Computer generated designs made by software other than FHWA's FoSSA computer program shall require verification that the computer program's design methodology meets the requirements provided herein; this shall be accomplished by either:

1. Provide complete, legible, calculations that show the design procedure step-by-step for the surcharge and PVD spacing. Calculations may be computer generated provided that all input, equations, and assumptions used are shown clearly.

2. Provide a diskette with the input files and the full computer output of the FHWA sponsored computer program FoSSA (latest version). This software may be obtained at:

ADAMA Engineering, Inc.
33 The Horseshoe, Covered Bridge Farms
Newark, Delaware 19711, USA
Tel. (302) 368-3197, Fax (302) 731-1001

19.2.2 Earthquake Drains

Earthquake (EQ) drains are a subset of PVDs that are used to mitigate/limit the effects of seismically induced liquefaction. While PVDs are thin plastic strips consisting of a rigid core sheathed in filter fabric; EQ drains are perforated, corrugated plastic pipe placed in a filter fabric sock. Earthquake drains can range in size from 1-1/2 to 10 inches in diameter, but are more typically 4 to 6 inches in diameter. Earthquake drains are used to reduce the excess pore pressures generated by a seismic event that can lead to liquefaction in loose granular soils (see Chapter 13 for a discussion of liquefaction). The theoretical background for earthquake drains is presented in *FEQDrain: A Finite Element Computer Program for the Analysis of the Earthquake Generation and Dissipation of Pore Water Pressure in Layered Sand Deposits with Vertical Drains*.

EQ drains work by reducing the pore pressure ratio (r_u, see equation 19-3), to a level that prevents or limits the potential for liquefaction. Recent research on the applicability of EQ drains has indicated that some liquefaction induced settlement will still occur. Typically a r_u of 0.65 is used to determine the spacing of the drains. However, because of the uncertainties in the amount of liquefaction induced settlement, the effect of high fines content (i.e., percent passing the No. 200 greater than 5 percent), and the effect of high accelerations caused by earthquakes in South Carolina, the r_u shall be limited to 0.50. Using an r_u of this magnitude will cause the drain spacing to become smaller and potentially increasing the drain size.

$$r_u = \frac{\Delta u}{\sigma'_v} \qquad \text{Equation 19-3}$$

Where,

r_u = Pore pressure ratio

Δu = Change in pore pressure

σ'_v = Effective overburden pressure

19.2.3 Construction Considerations

PVDs are installed using equipment similar in size and appearance to pile driving equipment and or foundation drilling equipment. A typical installation rig for PVDss is shown in Figure 19-2. The contractor is required to submit an installation plan, shop drawings, material samples, and anchorage details. A minimum 12-inch thick layer of clean sand is necessary at the top of the PVDs to provide a drainage path for release of the excess pore pressures. In some applications it will be appropriate to install strip drains across the ground surface to provide horizontal drainage at the top of the PVDs. The drainage layer many times can be installed as a part of the working platform necessary to make the site accessible to PVD installation equipment.

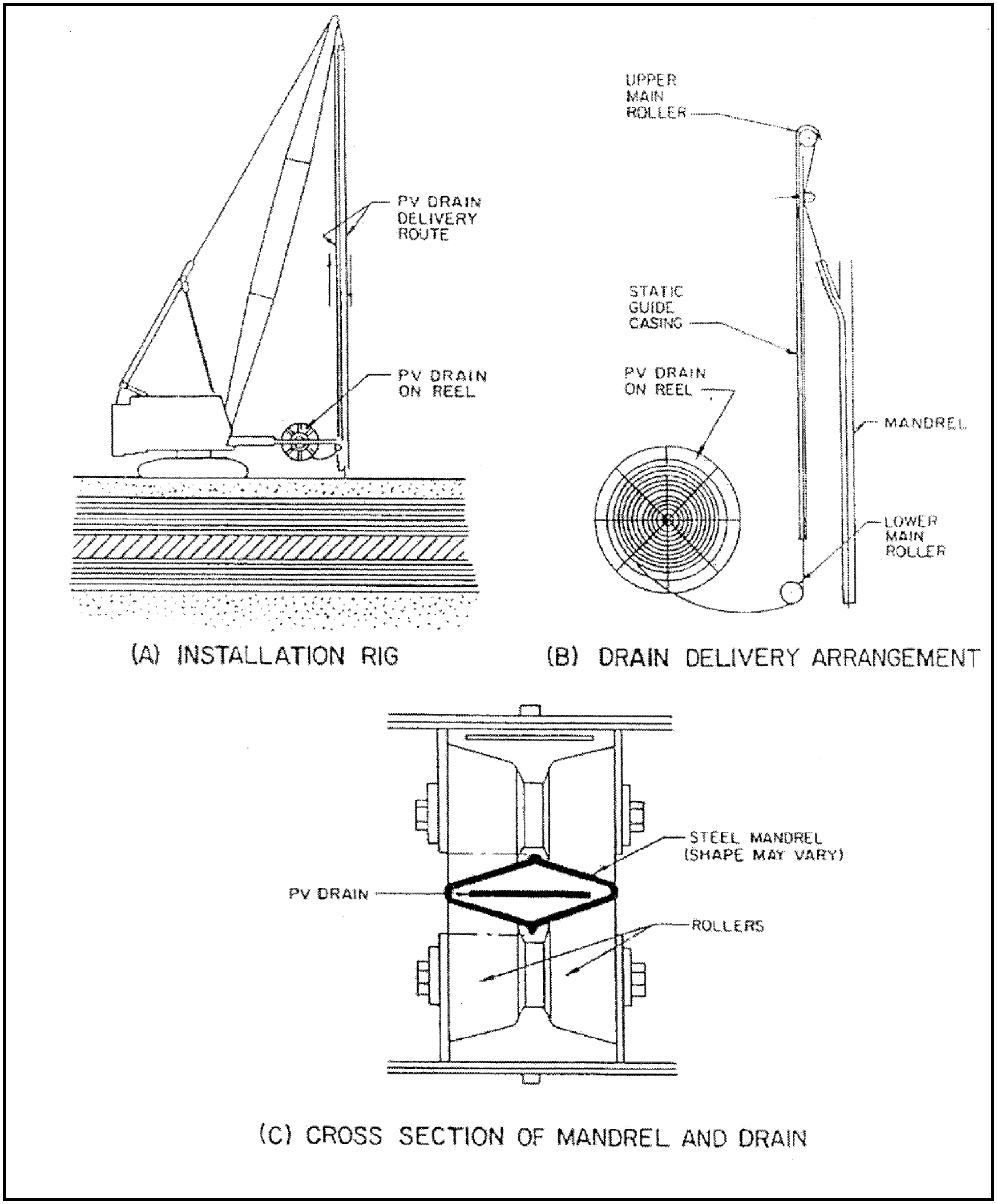


Figure 19-2, Crane Mounted Installation Rig
(Ground Improvement Methods – August 2006)

19.3 LIGHTWEIGHT FILL MATERIALS

Lightweight fill materials are used to limit settlement and increase stability through the use of materials with lower densities than conventional fill materials. Conventional fill materials (i.e., sand, silt and gravel) have densities that range from 115 to 140 pounds per cubic foot (pcf).

Lightweight fill materials can have densities ranging from 1 pcf for geofoam to 65 pcf for expanded clays and shales. In addition to reducing settlement and increasing stability, lightweight fill materials reduce the load applied to Earth Retaining Structures and increase an embankment's resistance to seismic loads by reducing the seismic inertial forces. Table 19-3 provides a list of lightweight fill materials used by SCDOT. Ground Improvement Methods provides additional lightweight materials; however, the use of these other lightweight fill materials, must be approved in writing by SCDOT (including the GDS, the PCS/GDS, and Office of Materials and Research (OMR))

Table 19-3, Lightweight Fill Materials
(modified from Ground Improvement Methods – August 2006)

Fill Material	Range of Density (pcf)	Range of Specific Gravity
Geofoam (EPS)	0.75 – 2.00	0.01 – 0.03
Foamed Concrete	20 – 60	0.3 – 0.8
Expanded Shale, Clay & Slate (ESCS)	37 – 65	0.6 – 1.0

19.3.1 General Applications and Limitations

19.3.1.1 Load Reduction

As indicated previously, one of the primary uses of lightweight fill is to reduce the load imposed on soft soils by normal weight fill materials. The use of lightweight fill materials reduces the driving forces, thereby increasing the overall global stability of the embankment or structure. A secondary effect of using lightweight fills is the reduction of the settlement under the imposed load. The amount of settlement reduction is directly proportional to the reduction in the load.

19.3.1.2 Shear Strength

According to Ground Improvement Methods:

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

19.3.1.3 Compressibility

> Many lightweight fill materials, such as foamed concrete and ESCS have a compressibility and elasticity similar to natural soils and rock. Under static loading, the amount of internal compression within the fill will often be similar to that for conventional earth fill materials. Under dynamic loading, the resiliency of the lightweight materials will often be similar to the natural soils. Geofoam compressibility or stress strain behavior is generally linear to stress levels of about 0.5 percent. Beyond that, yielding occurs and the material is subject to time-dependent creep."

19.3.1.4 Lateral Pressures

According to Ground Improvement Methods, "The lateral earth pressure at any depth is a function of the vertical overburden pressure multiplied by the coefficient of earth pressure and then reduced by the cohesion of the deposit. In the case of lightweight fills such as foamed concrete or geofoam, the cohesion of the material is high and the densities are low. *Each* of these factors tend to *significantly* reduce the amount of lateral earth pressure that is transmitted to adjacent structures such as retaining walls, tunnels or pile foundations below bridge abutments."

19.3.1.5 Drainage

ESCS materials like most of the granular lightweight fill materials have good drainage characteristics. Good drainage is beneficial behind a retaining wall to eliminate hydrostatic pressures.

19.3.1.6 Construction in Adverse Weather

According to Ground Improvement Methods:

> "It is difficult, if not impossible, to place and compact conventional soils during extremely cold or wet weather. However, geofoam, ESCE and foam concrete, have been successfully installed in *inclement* weather.

19.3.1.7 Seismic Considerations

> In Japan, there have been case histories where a highway embankment constructed of geofoam did not fail in a severe earthquake, even though adjacent sections of a soil embankment did. The lower unit weight of the material results in lower inertial forces under seismic loading."

19.3.1.8 Limitations

Lightweight fill materials have limitations for use; however, these limitations can be overcome by proper evaluation, design, and construction techniques. The following list of limitations is obtained from Ground Improvement Methods:

- "*Availability of the materials*. Certain geographic areas may have an abundance of one type of lightweight fill material, but not of another. Unless the lightweight fill material is available *locally*, the transportation costs raise the price considerably, and make these materials non-competitive.
- *Construction Methods*. In general, all lightweight fill materials involve some special procedures with regard to handling, transportation, placement and compaction. Some lightweight fill materials could be difficult to place and handle. Foam concrete requires the use of specialized equipment at the site to introduce air and other additives into the mixture before placement.
- *Durability of the fill deposits*. Some lightweight fill materials (e.g., geofoam) must be protected to ensure longevity. Because geofoam is subject to deterioration from

hydrocarbon spills, a concrete slab or geomembranes are generally placed over the surface of the blocks.

- *Environmental concerns*. Some lightweight fill materials generate leachate as water passes through these deposits. Fortunately, design methods have been developed to minimize the amount of leachate, and, to date, these measures have worked satisfactorily. However, the additional costs of these measures should be considered during design.
- *Geothermal properties*. Most lightweight fill materials possess geothermal properties that are different than soil. This can lead to accelerated deterioration of flexible pavements and/or problems with differential icing of pavement surfaces due to an alteration of the heat balance at the earth's surface. Essentially, most lightweight fill materials act as thermal insulation, even though this is not an intended or desirable function. However, this can be effectively controlled by placing a suitable thickness (20-inch, minimum) of soil and/or paving materials over the surface of the lightweight fill material."

19.3.2 Geofoam

According to Ground Improvement Methods, "Geofoam is a generic term used to describe any foam material used in a geotechnical application. Geofoam includes expanded polystyrene (EPS), extruded polystyrene (XPS), and glassfoam (cellular glass). Geofoam was initially developed for insulation material to prevent frost from penetrating soils." Geofoam materials have the advantage of being not only lightweight, but also may be cut to any size of shape to fit the requirements of the project. NCHRP Project 24-11, Guidelines for Geofoam Applications in Embankment Projects, contains detailed design guidelines for the use of EPS geofoam in roadway embankments and bridge approaches. Geofoam is a lightweight fill material that has a specific compressive strength.

According to Ground Improvement Methods:

> "The overall design process when using EPS geofoam is divided into three phases in order to consider the interaction between the three major components in the embankment.
>
> - Design to preclude external (global) *in*stability of the embankment. This should include considerations for settlement, bearing capacity, and slope stability under the projected loading conditions.
> - Design for internal stability within the embankment mass. The design must ensure that the geofoam mass can support the overlying pavement system without immediate and time dependent creep compression.
> - Design of an appropriate pavement system for the subgrade provided by the underlying geofoam blocks.
>
> External stability analyses generally follow traditional geotechnical procedures, although stress distribution must consider a non-homogeneous embankment. Stability analyses require modeling of undrained shear strength of geofoam, which presents some uncertainties. For shear strength, NCHRP 24-11 recommends the use of one-quarter of the compressive strength of the geofoam.

Internal stability analyses primarily consist of selecting an EPS geofoam type with adequate properties to support the overlying pavement system and the traffic load without excessive settlement of the surface. The current design approach is a deformation-based methodology using the total stress from all loads on the EPS blocks and elastic limit stress and the initial tangent modulus to evaluate load-induced deformations. Table *19-4* provides the minimum recommended values of elastic limit stress for various EPS densities." Table 19-5 summarizes the design parameters associated with the use of EPS geofoam.

Table 19-4, EPS Elastic Properties
(modified from Ground Improvement Methods – August 2006)

Material Designation	ASTM Designation	Dry Density each block (pcf)	Dry Density Test Specimen (pcf)	Elastic Limit Stress (psi)	Initial Tangent Young Modulus (psi)
EPS 40	I	1.00	0.90	5.8	580
EPS 50	VIII	1.25	1.15	7.2	725
EPS 70	II	1.50	1.35	10.1	1015
EPS 100	IX	2.00	1.80	14.5	1450

Table 19-5, EPS Geofoam Design Guidelines
(modified from Ground Improvement Methods – August 2006)

<table>
<tr><th colspan="4">Design Parameters</th></tr>
<tr><td>Density, dry</td><td>1 – 2 pcf</td><td>CBR</td><td>2 – 4</td></tr>
<tr><td>Compressive and Flexural Strength</td><td>6 – 14 psi[1]</td><td rowspan="2">Coefficient of Lateral Earth Pressure</td><td rowspan="2">Lateral pressures from adjacent soil mass may be reduced to a ratio of 0.1 of horizontal to vertical pressure</td></tr>
<tr><td>Modulus of Elasticity</td><td>580 – 1450 psi</td></tr>
<tr><th colspan="4">Environmental Considerations</th></tr>
<tr><td colspan="4">There are no known environmental concerns. No decay of the material occurs when placed in the ground.</td></tr>
<tr><th colspan="4">Design Considerations</th></tr>
<tr><td colspan="4">
EPS blocks will absorb water when placed in the ground. Blocks placed below water have resulted in densities of 4.8 – 6.4 pcf after 10 years, while blocks above the water had densities of 1.9 – 3.2 pcf for the same period. 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 resistance factor of 0.75 against uplift.
Because petroleum products will dissolve geofoam, a geomembrane or a reinforced concrete slab is used to cover blocks in roadways in case of accidental spills.
Differential icing potential of pavement, due to a cooler pavement surface above the EPS versus pavement above a soil only subgrade. Differential icing can be minimized by providing a sufficient thickness of soil between the EPS and top of the pavement surface.
Use side slopes flatter than or equal to 2H:1V and a minimum cover thickness of 1 foot. If a vertical face is needed, cover exposed face blocks with shotcrete or other material to provide long-term UV protection.
</td></tr>
</table>

[1]Varies with density

19.3.3 Foamed Concrete

Foamed concrete is created by introducing a preformed foam into a cement water slurry. The preformed foam is designed for concrete and creates a network of discrete air cells within the cement matrix. Sand and fly ash may be added to the mixture with the fly ash partially replacing a portion of the cement. After blending these materials to the specified density, the resulting slurry is pumped into place. Foamed concrete is unique for each application and is normally mixed on site. The quality of foamed concrete is monitored through the cast density. The compressive strength is directly related to the cast density of the mixture. Like geofoam, foamed concrete has a specific shear strength that is used in design.

According to Ground Improvement Methods:

> "Foam concrete is a liquid product that is practically self leveling, can be pumped over a distance as great as 3,300 feet, and will begin to harden between 2 to 6 hours after production. Design with this product is analogous to design with conventional concrete, using the typical average properties summarized in Table *19-7* for a range of cast densities. In computing buoyant stability, the low value of the volumetric mass should be used. For settlement calculations, the high value of the volumetric mass should be used including consideration for water absorption.
>
> *Table 19-6* summarizes key design considerations."

Table 19-6, Foamed Concrete Design Guidelines
(modified from Ground Improvement Methods – August 2006)

<table>
<tr><th colspan="4">Design Parameters</th></tr>
<tr><td>Density, dry</td><td>20 - 61 pcf</td><td>Freeze-thaw Resistance, 100 cycles</td><td>92 – 98 %[1]</td></tr>
<tr><td>Compressive Strength</td><td>45 - 360 psi[1]</td><td rowspan="2">Coefficient of Lateral Earth Pressure</td><td rowspan="2">Negligible for vertical loads applied directly over the foamed concrete. Lateral pressures from adjacent soil mass may be transmitted undiminished.</td></tr>
<tr><td>Water Absorption</td><td>1.4 – 15 psf[1]</td></tr>
<tr><th colspan="4">Environmental Considerations</th></tr>
<tr><td colspan="4">There are no known environmental concerns.</td></tr>
<tr><th colspan="4">Design Considerations</th></tr>
<tr><td colspan="4">Buoyancy could be a problem if foamed concrete is placed below the water table and there is not sufficient vertical confinement.
The lower compressive strength mixes are affected by freeze-thaw cycles. The product should be used below the zone of freezing or a higher compressive strength used. Densities greater than 37 pcf have reported excellent freeze-thaw resistance.
There is some absorption of water into the voids, which could affect the density and compressive strength. Saturation by water should be prevented by construction of a drainage blanket and drains.</td></tr>
</table>

[1]Varies with density

Table 19-7, Typical Properties of Foam Concrete (modified from Ground Improvement Methods – August 2006)

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

[1] Density when cured to constant mass at105° Celsius.
[2] Actual properties depend on mix designs, raw materials used, and manufacturing methods.
[3] Bending tensile strength and dynamic E-modulus determined with 2-point symmetrical bending test.
[4] With 4-point bending test.
[5] E_{dyn} as % of E_0 after 100 cycles according to ASTM C666, procedure B, modified for longer freeze/thaw cycle due to insulating properties of foam concrete.
[6] E_{dyn} as % of E_0 after 300 cycles according to ASTM C666, procedure B.
[7] K-value determined according to ASTM D 5084, method B, "falling head," using cell pressures or confining pressures of P_c = 2.5 psi & 18 psi.
[8] Total absorption in kg through 1 m^2 of foam concrete during 10 years exposure of that area to a constant head of 1 meter water.
[9] Heat conductivity (W/m.K) measurements are done with a Showa Denko K.K; the minimum value is valid for a moisture content of 70%, and the maximum value is valid for a R.H. of

19.3.4 Expanded Shale, Clay & Slate

Expanded shale, clay and slate (ESCS) is a granular lightweight fill material. In other words, the strength of these materials is based on the interlock between individual particles, similar to sands and gravels. ESCS is a synthetic aggregate created from heating certain shales, clays and slates in a rotary kiln to temperatures in excess of 1,800°F. During this process the clay minerals montmorillonite, illite, and kaolinite become completely dehydrated and expand. Once completely dehydrated, these materials will not re-hydrate under atmospheric conditions; therefore, retaining the expanded shape. The materials are graded through a screening process and may have rounded, cubical or sub-angular particle shapes. These particles are durable, chemically inert and relatively insensitive to moisture; however, the particles will absorb and retain some water. ESCS materials can be expensive to manufacture, which has led to the use of these materials primarily as lightweight aggregate in structural concrete.

The design procedures using ESCS use conventional geotechnical methods associated with granular soils. Table 19-8 summarizes key design considerations.

Table 19-8, ESCS Design Guidelines
(modified from Ground Improvement Methods – August 2006)

<table>
<tr><th colspan="5">Design Parameters</th></tr>
<tr><td rowspan="2">Dry Density</td><td>Compacted</td><td>50 – 65 pcf</td><td rowspan="2">Permeability</td><td rowspan="2">High</td></tr>
<tr><td>Loose</td><td>40 – 54 pcf</td></tr>
<tr><td rowspan="2">Angle of Shearing Resistance</td><td>Compacted</td><td>37° – 44°</td><td rowspan="4">Grain Size Gradation</td><td rowspan="4">5 – 25 mm</td></tr>
<tr><td>Loose</td><td>35°</td></tr>
<tr><td rowspan="2">Coefficient of Subgrade Reaction</td><td>Compacted</td><td>140 – 155 pci</td></tr>
<tr><td>Loose</td><td>33 – 37 pci</td></tr>
<tr><th colspan="5">Environmental Considerations</th></tr>
<tr><td colspan="5">There are no known environmental concerns.</td></tr>
<tr><th colspan="5">Design Considerations</th></tr>
<tr><td colspan="5">• The material will absorb some water after placement, when continually submerged. Samples compacted at a water content of 8.5 percent have been found after 1 year to have a water content of 28 percent. Over a longer period of time, the estimated long-term water content would be about 34 percent.
• Side slopes of embankments should be covered with a minimum of 3 feet of soil cover.
• Use side slopes of 1.5H:1V or flatter to confine the material and provide internal stability.
• For calculating lateral earth pressures, use an angle of shearing resistance of 35°.</td></tr>
</table>

19.4 VIBRO-COMPACTION

Vibro-compaction is a ground improvement method that uses a specialized vibrating probe for in-situ densification of loose sands at depths beyond that which surface compaction equipment is inadequate (see Figure 19-3). The vibrator densifies loose granular, cohesionless soils using mechanical vibrations and water to overcome the in-situ effective stresses between the soil grains causing the grains to rearrange under the action of gravity into a denser state. The vibrations in the immediate vicinity of the vibrator induce liquefaction of saturated loose granular, cohesionless soils. Generally, vibro-compaction can be used to achieve the following results:

- Increased soil bearing capacity
- Reduced foundation settlements
- Increased resistance to liquefaction
    - Compaction to stabilize pile foundations driven through loose granular materials
    - Densification for abutments, piers and approach embankment foundations
- Increased shear strength
- Reduced permeability
- Filling of voids in treated areas

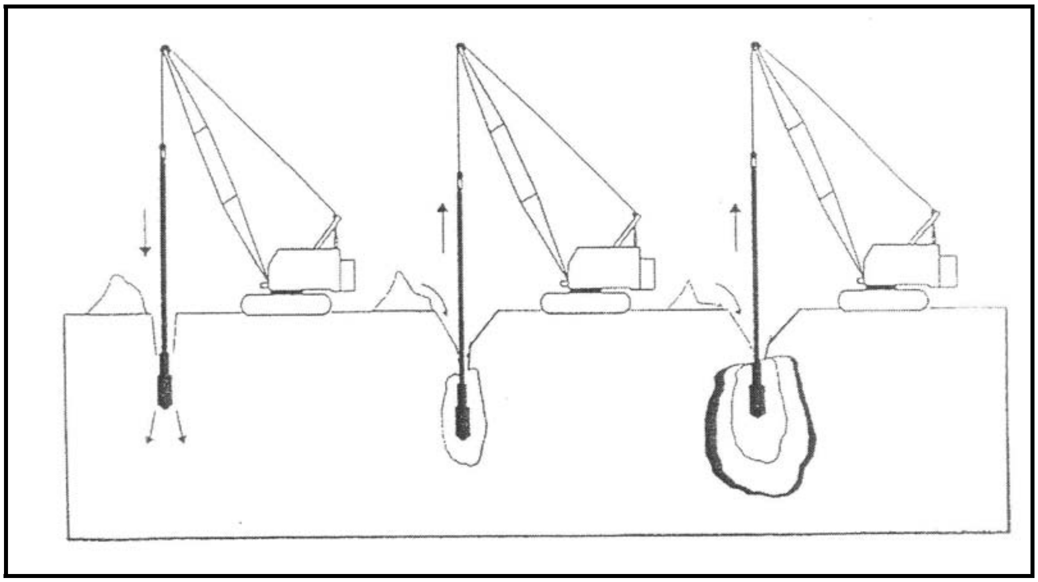


Figure 19-3, Vibro-Compaction
(Ground Improvement Methods – August 2006)

19.4.1 <u>Advantages and Disadvantages/Limitations</u>

19.4.1.1 Advantages

These advantages are described in, and come from, <u>Ground Improvement Methods</u>:

> "As an alternative to deep foundations, vibro-compaction is usually more economical and often results in significant time savings. Loads can be spread from the footing elevation, thus minimizing problems from lower, weak layers. Densifying the soils with vibro-compaction can considerably reduce the risk of seismically induced liquefaction. Vibro-compaction can also be cost-effective alternative to removal and replacement of poor load-bearing soils. The use of vibro-compaction allows the maximum improvement of granular soils to depths of up to 165 feet. The vibro-compaction system is effective both above and below the natural water level.

19.4.1.2 Disadvantages/Limitations

> The major disadvantage of vibro-compaction is that it is effective only in granular, cohesionless soils. The realignment of the sand grains and, therefore, proper densification generally cannot be achieved when the granular soil contains more than 12 to 15 percent silt OR more than 2 percent clay. The maximum depth of 165 feet may be considered a disadvantage, but there are very few construction projects that will require densification to a greater depth.
>
> Like all ground improvement techniques, a thorough soils investigation program is required. A more detailed soils analysis may be required for vibro-compaction than for a deep foundation project. This is because the vibro-compaction process utilizes the native soil to the full depth of treatment to achieve the end result. A comprehensive understanding of the total soil profile is therefore necessary. A vibro-compaction investigation will require continuous standard penetration tests (SPT) and/or cone penetrometer (CPT), as well as gradation tests to verify that the soils are suitable for vibro-compaction."

The potential environmental impact may be considered another disadvantage to vibro-compaction. The use of wet vibro-compaction, while efficient and used on the vast majority of projects, requires the use of water to jet the vibrator into the ground. The effluent from the jetting process requires at least temporary containment to allow any fine soil particles to settle out. Further, this method of ground improvement may not be acceptable if the existing subsurface environment, either soil or water is contaminated. If contamination is present, other ground improvement methods should be considered.

19.4.2 Design and Analysis

The design and analysis of vibro-compaction is based mainly on the grain-size distribution as shown in Figure 19-4. Soil compaction, as achieved in the vibro-compaction process through the rearrangement of soil particles, is not possible in cohesive, fine-grained soils. The cohesion between the particles prevents rearrangement and compaction from occurring. Soils on the coarse side of Zone B may be readily compacted using vibro-compaction. If the grain-size distribution curve falls in Zone C, it is advisable to backfill with gravel instead of sand during the compaction process. The use of gravel will improve the contact between the vibrator and the treated soil, drastically increasing compaction. Soils located partially or completely in Zone D are not suitable for vibro-compaction; however these soils (Zone D) are suitable for vibro-replacement (i.e. stone columns).

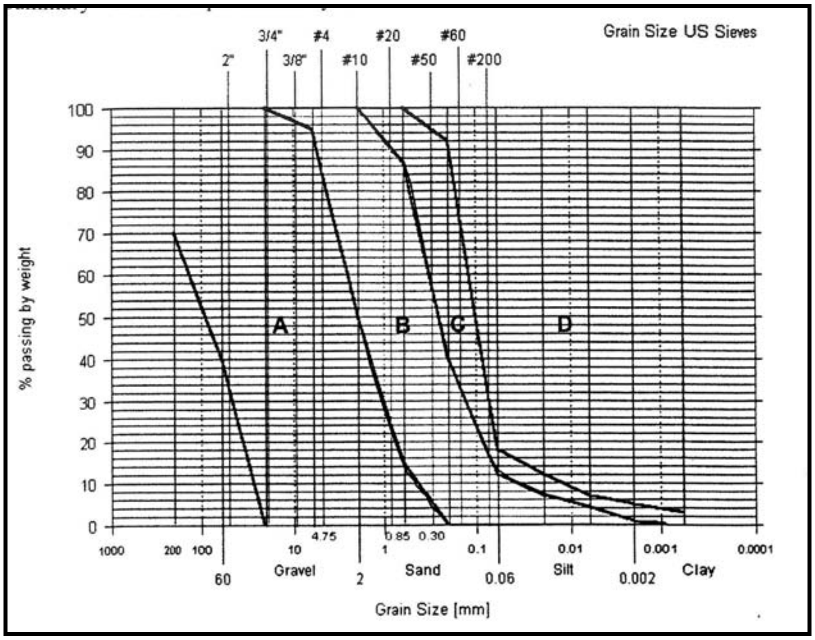


Figure 19-4, Soil Range Treatable by Vibro-Compaction (Ground Improvement Methods – August 2006)

As indicated previously, the vibrations induced by vibro-compaction cause the inter-granular forces acting between soil grains to reduce to zero allowing the soil particles to shift under the action of the vibrations and gravity into a more dense state. This more dense state has a reduced void ratio and correspondingly has a reduced compressibility and increase in the shearing resistance of the soil. The achievable reduction in void ratio depends on grain shape, soil composition (gradation), and vibration intensity. By controlling the advancement and withdrawal of the vibrator, a compact soil cylinder is formed. The diameter of the cylinder is based on the grain-size distribution, the initial soil density, and the vibrator characteristics. Typical vibrators have dynamic forces that range from 33,750 to 101,250 pounds with frequencies ranging from 1,800 to 2,300 revolutions per minute (rpm). For vibro-compactors operating at lower frequencies, better densification is usually produced. This is because low frequency vibrators usually have a higher amplitude, which translates into a greater compactive effort. Additionally, the natural frequency of most densifiable soils is closer to 1,500 rpm than to 3,000 rpm.

The increase in density of the granular soils causes a downward movement of the soil around the vibrator. This downward movement creates a conical depression at the ground surface. This depression requires constant filling with additional granular materials. A suitability number (S_N) is used to determine the suitability of a granular material as replacement material in vibro-compaction. The S_N is based on the settling rate of the backfill in water and experience. The S_N is determined using the following equation and the rating criteria are presented in Table 19-9. The backfill materials consist of sand or sand and gravel, with less than 10 percent by weight passing the #200 sieve and containing no clay.

$$S_N = 1.7\sqrt{\frac{3}{(D_{50})^2} + \frac{1}{(D_{20})^2} + \frac{1}{(D_{10})^2}} \qquad \text{Equation 19-4}$$

Where,

D_{50} = Grain size diameters for 50 percent passing in millimeters
D_{20} = Grain size diameters for 20 percent passing in millimeters
D_{10} = Grain size diameters for 10 percent passing in millimeters

Table 19-9, Backfill Evaluation Criteria
(modified from Ground Improvement Methods – August 2006)

S_N	0 – 10	11 – 20	21 – 30	31 – 40	> 41
Rating	Excellent	Good	Fair	Poor	Unsuitable

19.4.2.1 Preliminary Design

The vibrator is hung from a crane cable or, in some instances; it is mounted to leads in a similar fashion as foundation drilling equipment. The vibrator penetrates under its self weight (or crowd of the machine if mounted in leads) and, at times, the action of water jets. The vibration and water imparted to the soils transforms the loose soils to a more dense state. Generally vibro-compaction would be used to achieve the following results on SCDOT projects:

- Increase soil bearing capacity
- Reduce immediate foundation settlement
- Increase resistance to liquefaction
- Increase shear strength

As indicated previously, the primary purpose of vibro-compaction is to increase the in-situ density of granular soils. Density is a measure of the unit weight of the soil; however, obtaining the unit weight of in-situ is extremely difficult. Therefore, the relative density (D_r) is used as measure of the increase or decrease in density of a soil. Relative density is defined in the following equation.

$$\boldsymbol{D_r} = \frac{\gamma_n - \gamma_l}{\gamma_d - \gamma_l} \times \frac{\gamma_d}{\gamma_n} \times \mathbf{100\%} \qquad \text{Equation 19-5}$$

Where,

γ_n = Dry density of the soil in-situ

γ_l = Dry density of the soil in its loosest state

γ_d = Dry density of the soil in its densest state

Relative density has been correlated over the years in geotechnical practice. Chapter 6 also includes correlations of D_r versus SPT N-values. Table 19-10 provides the relationship between D_r and various field tests. Higher D_r equates to an increase in bearing capacity and a corresponding reduction in settlement. The resistance to liquefaction increases with increasing D_r and the active earth pressure on an ERS decreases while the passive earth pressure on an ERS increases. According to Ground Improvement Methods, “With vibro-compaction, the angle of internal friction is increased on average 5 to 10 degrees, resulting in much higher shear resistance.”

Table 19-10, Penetration Resistance and Sand Properties (modified from Ground Improvement Methods – August 2006)

Description	Very Loose	Loose	Medium Dense	Dense	Very Dense
SPT N-values blows per foot	< 4	5 – 10	11 – 30	31 – 50	> 51
CPT Tip Resistance tsf	< 51	51 – 102	102 – 154	154 – 205	> 205
D_r %	< 15	16 – 35	36 – 65	66 – 85	86 - 100
Dry Unit Weight pcf	< 89	89 - 102	102 - 115	115 - 127	> 127
CSR[1]	< 0.04	0.04 – 0.12	0.12 – 0.33	0.33 – 0.40	-
Shear Wave Velocity, V_s fps	< 394	395 - 525	526 – 656	657 - 738	>739

[1]CSR – Cyclic Stress Ratio Causing Liquefaction

D_r is not only affected by the gradation of the soil, but also the area influenced by each compaction point. Figure 19-5 provides an example of an approximate relationship between D_r, soil type and treatment area for a specific vibrator. The increase in D_r is limited to 85 percent since, at this density, the improvement to the soil is enough to increase bearing and resistance to liquefaction and reduce settlement. It should be noted that a vibro-compaction contractor on an SCDOT project shall be required to present a similar type chart.

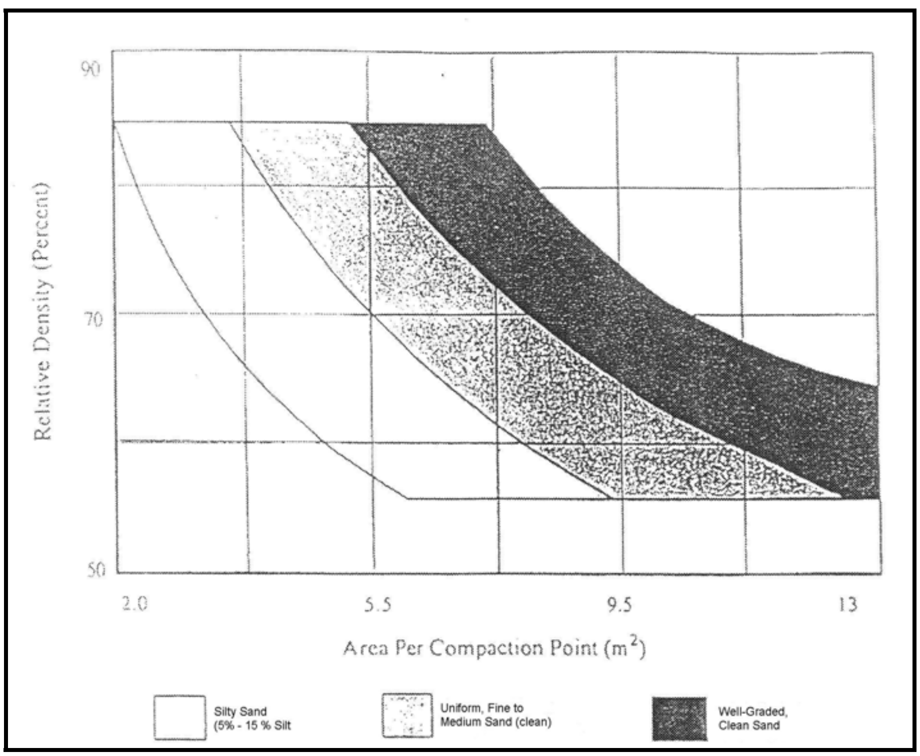


Figure 19-5, Variation of D_r with Tributary Area (Ground Improvement Methods – August 2006)

In South Carolina, vibro-compaction is primarily used to densify sites that have the potential for liquefaction (see Chapter 13). The improvement of the liquefiable soil should extend to the anticipated bottom of the liquefiable layer and should extend laterally to a distance at least equal to the depth of vibro-compaction as measured from the existing ground surface. Improvement for reducing lateral deformations of embankments is more effective when the foundation is treated in a zone between the crest and the toe of the embankment. Field performance suggests that the effect on structures will be minor when the supporting ground is improved to the “no liquefaction” side of the liquefaction potential curves (see Chapter 13).

According to Ground Improvement Methods:

> “A typical vibro-compaction program is designed with various probe spacing and patterns. The distance between compaction points is critical, as the density generally decreases as the distance from the probe increases. Stronger vibroprobes allow for wider spacing under the some soil conditions.
>
> The area compaction point pattern affects the densification. An equilateral triangular pattern is primarily used to compact large areas, since it is the most efficient pattern. The use of a square pattern instead of an equilateral triangular pattern requires 5 to 8 percent more points to achieve the same minimum densities in large areas.
>
> Given the in-situ soil gradation and relative density required, the spacing of compaction points can be determined. Figures *19-6 and 19-7* show typical area patterns and spacing for 80 percent relative density requirements. The spacing of the vibro-compaction points would be wider for lower relative density requirement.”

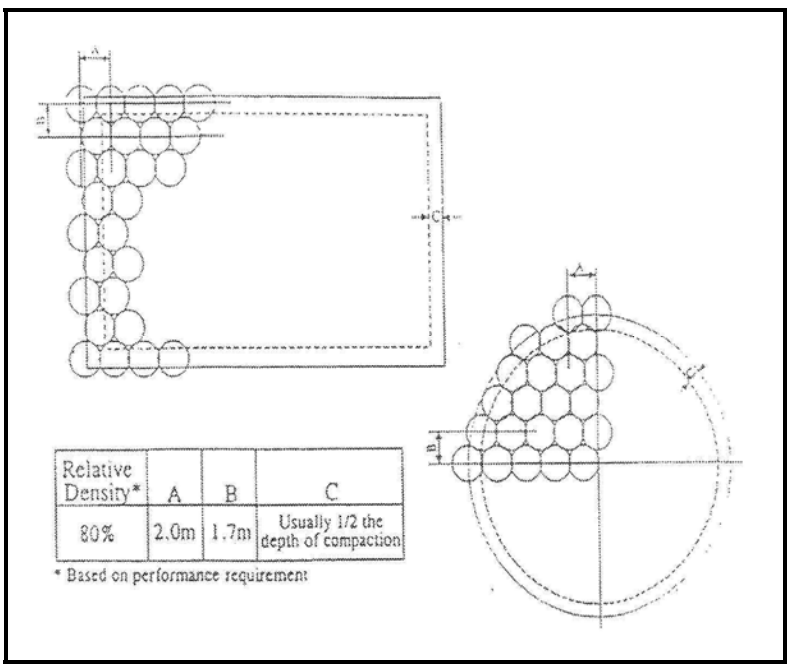


Figure 19-6, Typical Compaction Point Spacing for Area Layouts (Ground Improvement Methods – August 2006)

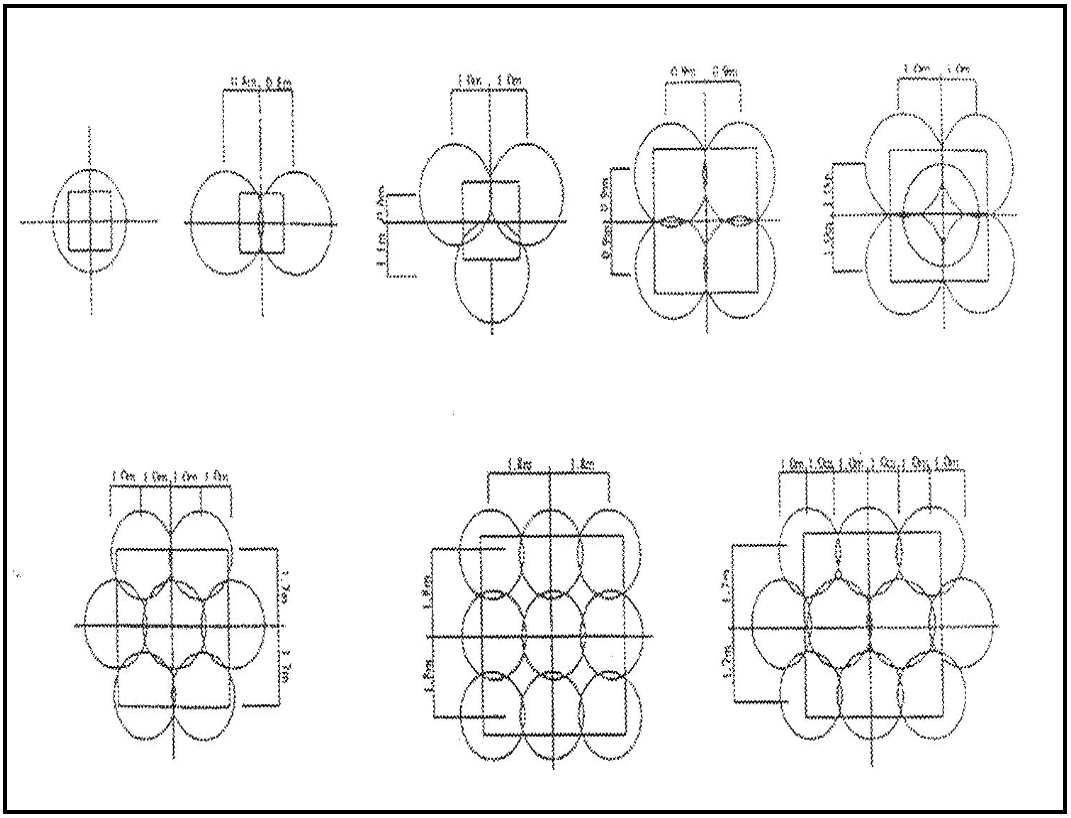


Figure 19-7, Typical Compaction Point Layouts for Column Footings (Ground Improvement Methods – August 2006)

19.5 STONE COLUMNS

Stone columns are constructed using vibratory methods similar to those methods used in vibro-compaction (see previous Section). The main differences is instead of using coarse-grained materials to simply fill the void created by the vibro-compaction, stone or other materials are placed to form a structural element (i.e., a column). Included in this Section along with stone columns are vibro-concrete columns (VCCs), geotextile-encased columns (GECs), and Geopier® Rammed Aggregate Pier^{TM} (Geopiers). Stone columns are constructed using either vibro-replacement or vibro-displacement. Table 19-11 provides definitions for both terms.

Table 19-11, Vibro-replacement and Vibro–displacement Definitions (Ground Improvement Methods – August 2006)

Vibro-replacement	Refers to the wet, top feed process in which jetting water is used to aid the penetration of the ground by the vibrator. Due to the jetting action, part of the in-situ soil is washed to the surface. This soil is then replaced by the backfill material.
Vibro-displacement	Refers to the dry, top or bottom feed process; almost no in-situ soil appears at the surface, but is displaced by the backfill material.

Stone columns are 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 19-8.

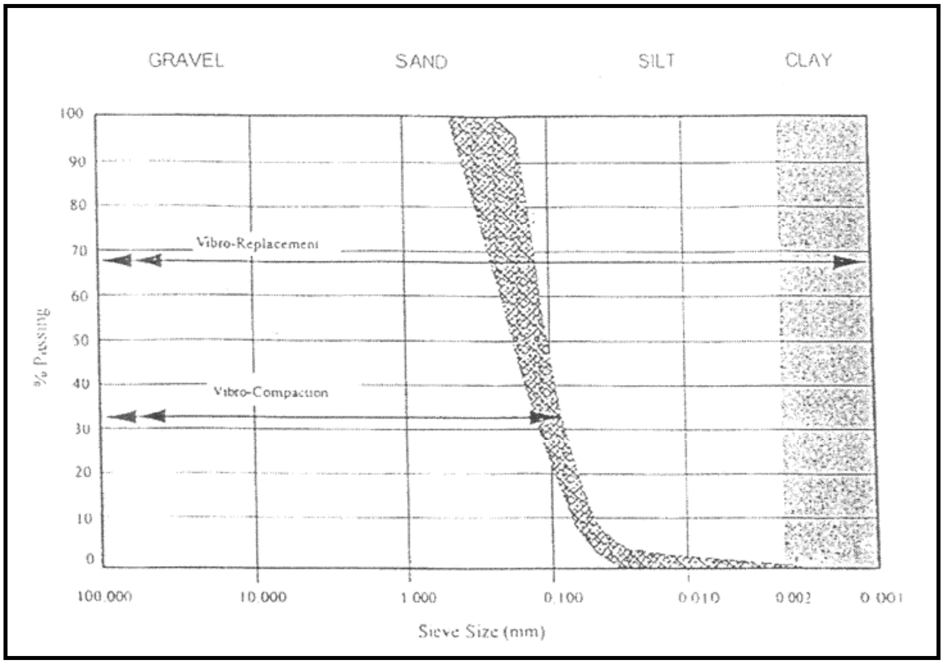


Figure 19-8, Applicable Grain-Size Distributions for Stone Columns (Ground Improvement Methods – August 2006)

As indicated previously, stone columns may be constructed using top or bottom feed methods (see Figures 19-9 and 19-10, respectively). The top feed method is a wet method and replaces the in-situ soil (i.e., vibro-replacement) with the stone column. In this method a high-pressure water jet is used to open a hole for the vibro-probe to follow into. Once the tip elevation is obtained the vibro-probe is retracted and stone is then placed into the hole from the top. The vibro-probe is then turned on and inserted into the stone to densify the stone, then the vibro-probe is retracted again and the process repeated until the stone column is formed. This method is used at sites with soft to firm soils with undrained shear strengths of 200 to 1,000 psf and a high groundwater table. When environmental impacts are anticipated, stone columns should be constructed using the vibro-displacement method. The vibro-displacement is a dry method that is either top or bottom feed. Using the oscillations of the vibrator in conjunction with the deadweight of the vibrator, air jetting and/or pre-augering, the vibrator is inserted into the ground without the use of jetting water. The top feed method can be used for short stone columns; however, for deeper columns and where the potential for hole collapse exists, the bottom feed method is used.

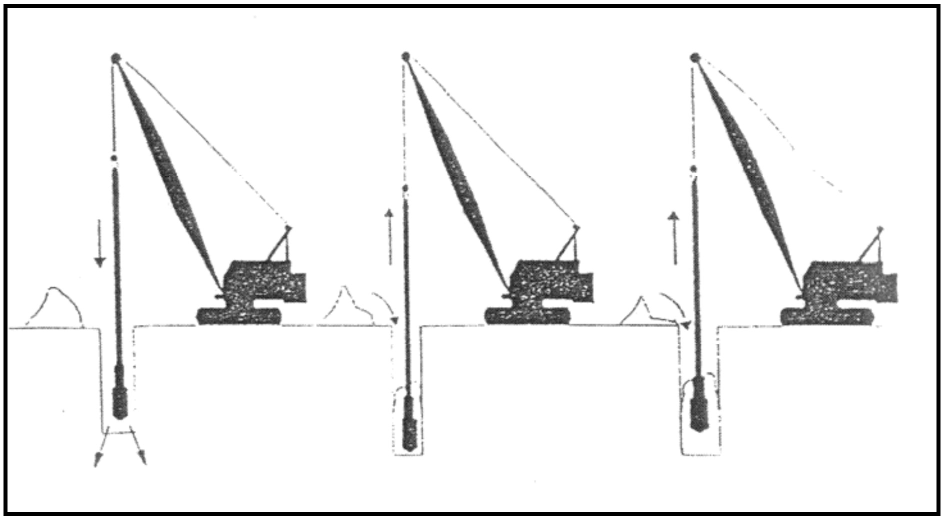


Figure 19-9, Top Feed Construction Method (Ground Improvement Methods – August 2006)

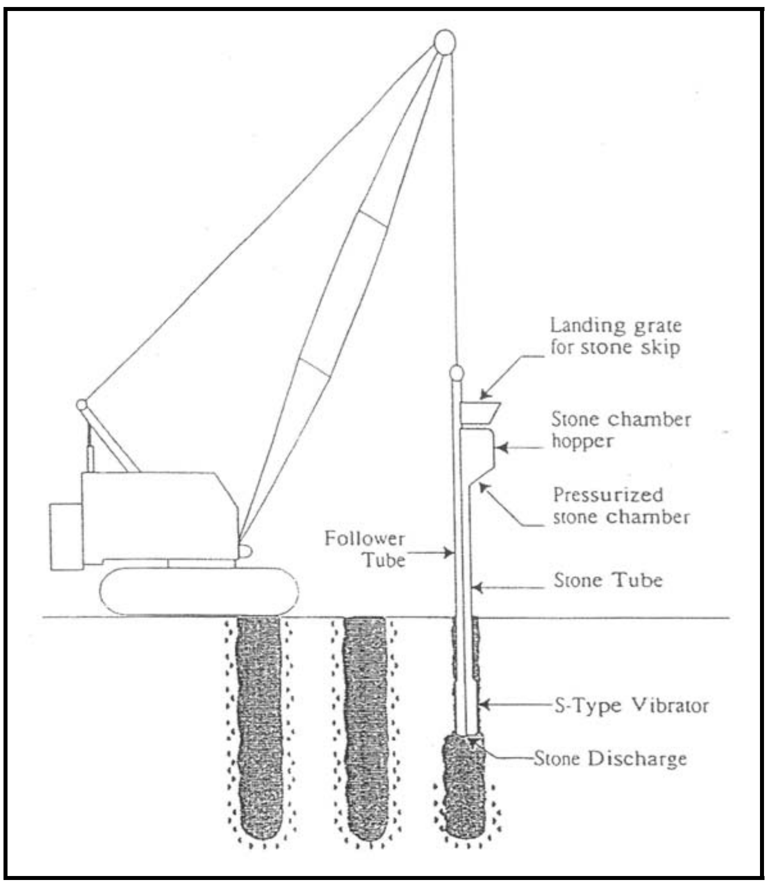


Figure 19-10, Bottom Feed Construction Method (Ground Improvement Methods – August 2006)

According to Ground Improvement Methods, "Since stone columns derive their strength and settlement characteristics from the surrounding soil, they do not perform well in very soft clay or peat with a thickness greater than the diameter of the column. Vibro-concrete columns *(VCCs)* were developed to treat these soils. Instead of feeding stone to the tip of the vibrator, concrete is pumped through an auxiliary tube to the bottom of the hole. This method can offer ground improvement advantages of the vibro-systems, with the load carrying characteristics of a deep foundation.

"The vibro-concrete column process employs a bottom feed vibrator that can penetrate the soils to a level suitable for bearing. Concrete is pumped through the vibrator assembly during initial withdrawal. The vibrator then repenetrates the concrete, displacing it into the surrounding soil to form a high-capacity, enlarged column base. The vibrator is then slowly withdrawn as concrete is pumped *and* maintained *at a* pressure to form a continuous shaft of concrete up to the ground level. At ground level, a slight mushrooming of the concrete column is constructed to assist the transfer of the applied loading into the vibro-concrete column (*see Figure 19-11*)."

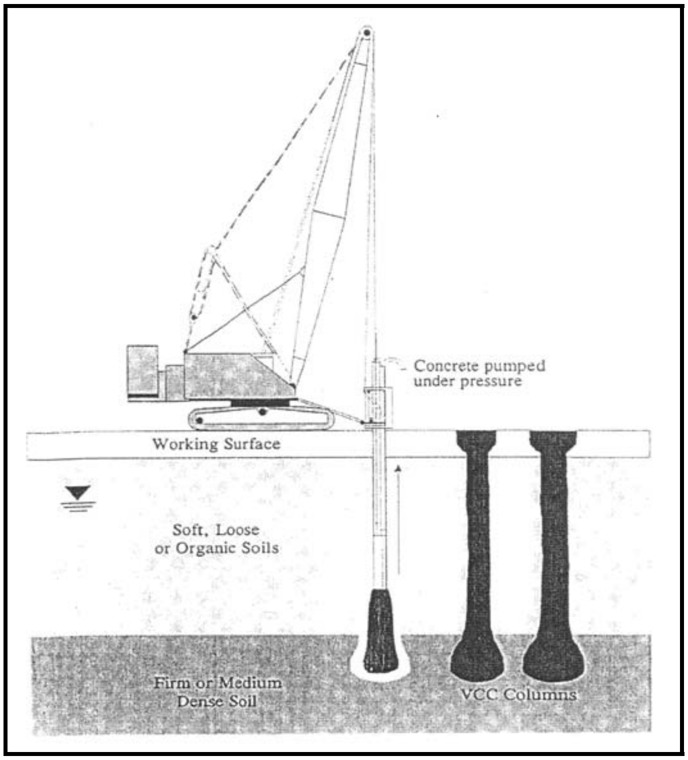


Figure 19-11, Vibro-Concrete Column
(Ground Improvement Methods – August 2006)

Geotextile-encased columns (GECs) consist of inserting continuous, seamless, high strength geotextile tubes into soft soil with a mandrel. The tube is then filled with either sand or fine gravel to form a column with a high bearing capacity. GECs typically have a diameter of 30 inches. GECs can be installed using either the replacement or the displacement methods. The replacement method consists of driving an open ended steel pipe pile to the bearing stratum. The soil within the pile is removed with an auger and the tube is inserted into the pile and then filled with sand or fine gravel. The displacement method use a steel pipe with two base flaps (the flaps close on contact with the ground surface) is vibrated to the bearing layer, displacing the soft soil. The geotextile casing is installed and filled with sand or fine gravel and the steel pipe pile is vibration extracted. During this process the sand or gravel within the geotextile is densified.

Geopier® Rammed Aggregate Pier™ (Geopiers) are a variant of stone columns, in that a 2- to 3-foot diameter hole is drilled into the foundation soil and gravel is added and then rammed into the foundation soils (see Figure 19-12). Geopiers typically extend to depths of 6 to 33 feet.

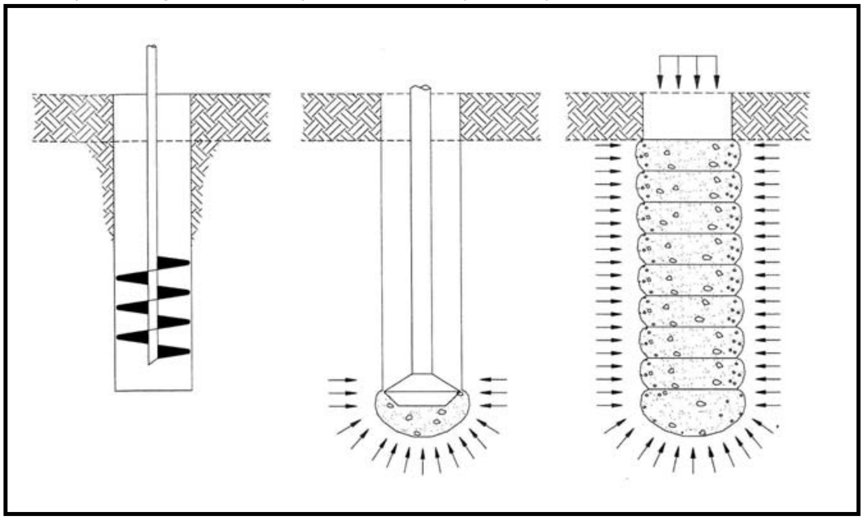


Figure 19-12, Geopier® Rammed Aggregate Pier™ (Ground Improvement Methods – August 2006)

Geopiers are most applicable in soft to stiff cohesive soils with undrained shear strengths ranging from 300 to 4,000 psf and in loose to medium dense silty and clayey sands. The soil must be stable without internal support (i.e., casing). The gravel is placed in relatively thin lifts with the first lift of gravel forming a bulb at the bottom of the pier, thus pre-stressing and pre-straining the soil beneath and around the bottom of the pier. The ramming process use a high-energy (250 to 650 kip-foot per foot) beveled tamper that both densifies the gravel and forces the gravel laterally into the sidewalls of the hole. This action increases the lateral stress in the surrounding soil, further stiffening the stabilized composite soil mass.

19.5.1 General Considerations

Stone columns can be used to improve the stability of slopes, increase bearing capacity, reduce total and differential settlements, and decrease the time for these settlements to occur, and to mitigate potential for liquefaction. Stone columns can be used to improve the stability of a slope by creating discrete zones of high strength material that will provide more resisting force along the potential failure surface. Stone columns can also increase the bearing capacity by transferring the load to a deeper, stronger layer and by densification of the in-situ soils through the use of vibro-displacement methods of installation. Further, stone columns can be used to reduce the amounts of total and differential settlement that a new embankment or a widened embankment would undergo without the improvement. The stone columns will also provide a conduit for the flow of ground water, thus decreasing the time for settlement to occur similarly to PVDs. Lastly, stone columns are used to mitigate the potential for liquefaction through densification of the in-situ materials and by providing pore pressure relief zones, because the stone column will have a greater hydraulic conductivity than the in-situ sands.

The advantages of stone columns are economy and technical feasibility to replace deep foundations with shallow foundations. Stone columns also provide a less expensive option to cut and replace, particularly on large sites with shallow groundwater. In developed areas where high-vibration methods such as dynamic compaction, deep blasting, or pile driving would have an impact on adjacent properties, low-vibration stone columns may provide a viable alternative to ground improvement. The use of stone columns could decrease the time required for construction by allowing construction to proceed immediately instead of waiting for the placement of surcharge. In areas that have a potential for liquefaction, the installation of stone columns can improve the cyclic resistance ratio (see Chapter 13). In addition, stone columns can provide vertical drainage and storage capacity to dissipate excess pore pressures induced by a seismic event. Geopiers have similar advantages to stone columns.

VCCs have the advantage of transferring loads similar to piles, while mobilizing the full ground improvement potential of a vibro-system. The installation of VCCs is a quiet process and induces minimal vibrations into the in-situ soils allowing for installation immediately adjacent to existing structures. Since this is a dry displacement process, there is no spoil to remove and no water requiring detention. VCCs have the additional advantage of being able to extend through thick very soft clays and organic materials.

According to Ground Improvement Methods, "The major advantage of GECs over stone columns is that they may be used in soft soils with undrained shear strengths as low as 25 psf. The geotextile provides the lateral constraint that the surrounding soils must provide for stone columns. GECs provide excellent vertical drainage, which may result in very rapid construction, due to the dissipation of pore water pressure."

The major disadvantage of stone columns is that stone columns are not effective in soils having thick layers of soft clays and organic materials. If the thickness is more than the diameter of the stone column, then stone columns may not be appropriate because the soft soils will not provide adequate lateral support of the stone column. In addition, stone column construction can be hampered by the presence of dense overburden, boulders, cobbles or other obstructions that may require pre-drilling prior to installation of the stone column. The major disadvantage of GECs and Geopiers is both methods rely on proprietary, patented technologies.

19.5.2 Feasibility

According to Ground Improvement Methods:

> "The degree of densification resulting from the installation of vibro-systems is a function of soil type, silt and clay content, soil plasticity, pre-densification relative densities, vibrator type, stone shape and durability, stone column area, column spacing, and compaction energy applied. Experience has shown that soils with less than fifteen percent passing the #200 (<0.074 mm) sieve, and clay contents less than two percent will densify due to the vibrations. Clayey soils do not react favorably to the vibrations, and the improvement in these soils is measured by the percent soil replaced and displaced by the stone columns, VCC, GEC, or Geopier.
>
> A generalized summary of the factors affecting the feasibility of stabilizing soft ground with stone columns is as follows:
>
> 1. The allowable design loading of a stone column should be relatively uniform and limited to a maximum of 112.5 kips per column, if sufficient lateral support by the in-situ soil can be developed.

2. The most significant improvement is likely to be obtained in compressible silts and clays occurring within 3.3 feet (10 m) of the surface and ranging in shear strength from 300 to 1000 psf.

3. Stone columns should not be used in highly sensitive soils *(see Chapter 7)*. Special care must be taken when using stone columns in soils containing organics and peat lenses or layers with undrained shear strength less than 200 psf. Because of the high compressibility and low strength of these materials, little lateral support may be developed and large vertical deflections of the columns may result. When the thickness of the organic layer is greater than one to two stone column diameters, the ability to develop consistent column diameters becomes questionable.

4. Ground improvement with stone column reduces settlements typically from thirty to fifty percent of the unimproved ground response and differential settlement from five to fifteen percent of unimproved soil response.

5. Stone columns have been used in clays having minimum (not average) undrained shear strengths as low as 150 psf. Due to the development of excessive resistance to penetration of the vibrator and economic considerations, a practical upper limit is in the range of undrained shear strength of 1,000 to 2,000 psf. Clays with greater shear strengths may, in fact, be strong enough to withstand the load without ground improvement.

6. Individual stone columns are typically designed for a bearing load of 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 stability of slopes and embankments. The design is usually based on conventional slip circle or wedge analyses utilizing composite shear strengths.

8. *The following relationship is recommended to prevent piping of the soil surrounding the stone column:*

$$20D_{S15} < D_{G15} < 9D_{S85} \qquad \textbf{\textit{Equation 19-6}}$$

Where,

D_{S15} = Diameter of the surrounding soil passing 15 percent
D_{G15} = Diameter of stone (gravel) passing 15 percent
D_{S85} = Diameter of the surrounding soil passing 85 percent

A generalized summary of the factors affecting the feasibility of stabilizing soft ground with VCC follows:

1. The allowable design load for VCC is a function of the diameter of the column, the allowable strength of the concrete, and the strength of the bearing layer. Typical

column diameters range from 18 to 24 inches. Typical allowable design loads for VCC range from 75 to 100 tons.

2. VCC are typically used in very soft clay and organic soils.

3. Typical VCC lengths vary from 16 to 33 feet.

A generalized summary of the factors affecting the feasibility of stabilizing soft ground with GEC follows:

1. GEC may be installed in soft, compressible clays up to depths of approximately 33 feet. Typical column diameters range from 2 to 3 feet.

2. GEC allowable load capacity is 20 to 40 tons.

3. Settlement of GEC typically occurs during construction of embankment and may be up to 10 to 20 inches.

A generalized summary of the factors affecting the feasibility of stabilizing soft ground with Geopiers follows:

1. The allowable design load for a Geopier is typically in the range of 25 to 75 tons per pier, depending on the lateral confinement provided by the surrounding soils (i.e., undrained shear strength $\geq$ 300 psf for soft saturated clays and SPT N-value $\geq$ 1 *blow per foot* for cohesionless soils).

2. Geopiers have been used effectively in soft soils, provided that top-of-pier stresses are lower than those needed to initiate pier bulging into the soft soils.

3. The installation of Geopiers in soils that do not stand open during drilling (loose granular soils, very soft cohesive soils) often requires the use of temporary casing, which reduces the installation rate and increases the cost of the piers.

4. The maximum practical depth of Geopiers is limited to 33 feet."

19.5.3 Environmental Considerations

Vibro-replacement methods use water jets to create a hole for the vibro-probe. The jetted water can cause the fine portions of the in-situ soils to come to the ground surface. The fines laden soil has to be contained temporarily to allow for sediment deposition. The resulting deposited material has to be disposed of properly. Further, this method may also bring other contaminants to the ground surface, causing the treatment and proper disposal of not only the sediments, but also the water used for jetting. For these reasons, the use of dry vibro-displacement methods is preferred for the installation of stone columns.

19.5.4 Design Considerations

The design of stone columns is still an empirical process; however, general design guidelines have been developed and are provided below. Additional information may be obtained from the following references.

1. Design and Construction of Stone Columns, Volume I, FHWA/RD-83/026
2. "The Design of Vibro Replacement," *Ground Engineering*

For stone columns to adequately perform, the soils surrounding the columns must provide sufficient lateral support to prevent bulging failures. In addition, the columns should terminate in a dense formation to prevent bearing failures. Stone columns are typically stiffer than the materials that surround the columns; therefore, the columns will settle less and will carry a larger portion of the applied load. The applied load is transferred between columns through soil arching. Ultimately equilibrium is reached when sufficient load has been transferred to the columns to prevent further settlement of the surrounding soils. In stability and bearing analyses, composite shear strength of the soil-stone column matrix is used. The composite shear strength is based on the shear strength of the in-situ soils, the shear strength of the stone materials, the area replacement, and stress ratios.

19.5.4.1 Unit Cell Concept

According to Ground Improvement Methods, "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 *19-13* and *19-14*. Although the tributary area forms a regular hexagon about the stone column, it can be closely approximated as an equivalent circle having the same total area. The resulting equivalent cylinder of material having a diameter (D_e) enclosing the tributary soil and one stone column is known as the "unit cell". The stone column is concentric to the exterior boundary of the unit cell.

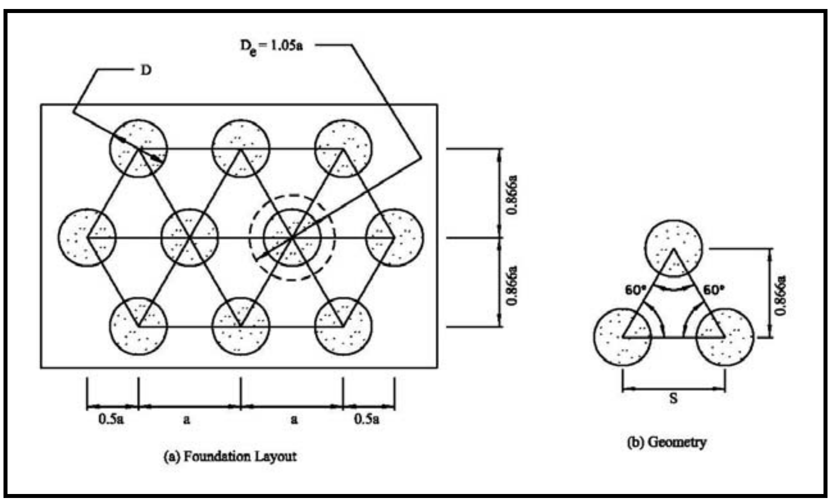


Figure 19-13, Stone Column Equilateral Triangular Pattern (Ground Improvement Methods – August 2006)

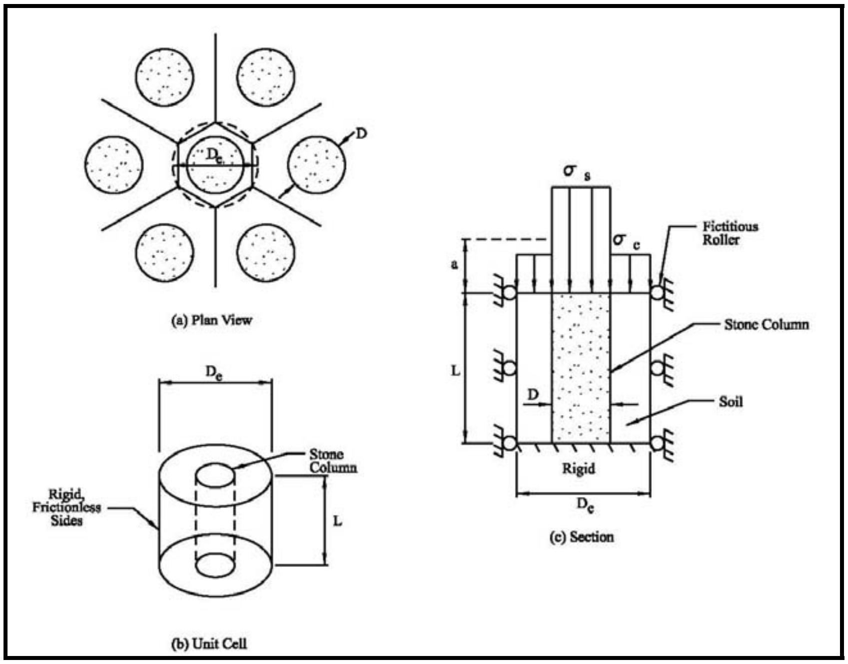


Figure 19-14, Unit Cell Idealization
(Ground Improvement Methods – August 2006)

19.5.4.2 Area Replacement Ratio

The Area Replacement Ratio (α_s) defines the area of the soil replaced by the stone column as a function of the tributary area of the unit cell to the area of the stone column. The more soil replaced by the stone column, the greater the effect on performance. Typical values of α_s range from 0.10 to 0.40.

$$\alpha_s = \frac{A_s}{A} \qquad \text{Equation 19-7}$$

$$a_s = \frac{1}{\alpha_s} = \frac{A}{A_s} \qquad \text{Equation 19-8}$$

Where,

α_s = Area replacement ratio
A_s = Area of the stone column
A = Total area within the unit cell
a_s = Area improvement ratio

19.5.4.3 Spacing and Diameter

According to Ground Improvement Methods, “Stone column diameters vary between 1.5 and 4.0 feet, but are typically in the range of 3.0 to 3.5 feet for the dry method *of installation*, and somewhat larger for the wet method *of installation*.

"Triangular, square or rectangular grid patterns are used with center-to-center column spacing of 5.0 to 11.5 feet. For footing support, *the stone columns* are installed in rows or clusters. For both footing or wide area support, *the stone columns* may extend beyond the loaded area."

19.5.4.4 Stress Ratio

The transfer of the applied load to the stone columns from the in-situ soils depends on the relative stiffness of the stone column to the in-situ soils, as well as the spacing and diameter of the stone columns. Because the stone columns and the in-situ soils deflect (strain) approximately equally, the stone columns must be carrying a greater portion of the load (stress) than the in-situ soils. This concept has also been called the equal strain assumption. This concept has been proven by both field measurements, as well as finite element analysis. The relationship between the stress in the stone column and the stress in the in-situ soil is defined in the following equation:

$$\boldsymbol{n} = \frac{\sigma_{\boldsymbol{s}}}{\sigma_{\boldsymbol{c}}} \qquad \text{Equation 19-9}$$

Where,

n = Stress ratio or stress concentration

σ_s = Stress in the stone column

σ_c = Stress in the surrounding soil

Measured values of n have generally been between 2.0 and 5.0. The theory indicates that n should increase with time. A high n-value (3 to 4) may be required in very weak soils and the column spacing is tight. Lower values of n (2 to 2.5) are required when the surrounding soil is stronger and the column spacing is wider. For preliminary design, a conservative n-value of 2.5 should be assumed.

Equilibrium of vertical forces for a given α_s is provided by the following equation.

$$\boldsymbol{q} = \sigma_{\boldsymbol{s}}\alpha_{\boldsymbol{s}} + \sigma_{\boldsymbol{c}}(\mathbf{1} - \alpha_{\boldsymbol{s}}) \qquad \text{Equation 19-10}$$

Where,

q = Average stress on the unit cell

The stresses in the stone column and the surrounding soil in the unit cell can be determined by rearranging the above equation.

$$\sigma_{\boldsymbol{c}} = \frac{\boldsymbol{q}}{[\mathbf{1} + (\boldsymbol{n} - \mathbf{1})\alpha_{\boldsymbol{s}}]} \qquad \text{Equation 19-11}$$

$$\sigma_{\boldsymbol{s}} = \frac{\boldsymbol{nq}}{[\mathbf{1} + (\boldsymbol{n} - \mathbf{1})\alpha_{\boldsymbol{s}}]} \qquad \text{Equation 19-12}$$

19.5.5 Verification

According to Ground Improvement Methods, "In-situ testing to evaluate the effect of the stone column construction on the native cohesive soil can be also specified. However, the specified test method should be selected on the basis of its ability to measure changes in lateral pressure in cohesive soils. The electric cone penetrometer *test* (CPT), the flat plate dilatometer *test* (DMT) and the pressuremeter *test* (PMT) appear to provide the best means for measuring the change, if any, in lateral stress due to stone column construction."

19.6 DYNAMIC COMPACTION

Dynamic compaction is the process of ground improvement using weights dropped from a height resulting in the application of high energy levels to the in-situ soil resulting in improvement of the soil. Typically, the weight (called a tamper) ranges from 11 to 39.6 kips and is dropped from heights of 30 to 100 feet. Dynamic compaction can typically be performed using conventional construction equipment as long as the crane has a free spool attached to allow the cable to unwind with minimal friction. The depth of improvement generally ranges from 10 to 36 feet for light- and heavy-energy applications, respectively. The light-energy applications consist of low weights and low drop heights, while heavy-energy applications consist of heavy weights dropped from high heights. Figure 19-15 provides a schematic of dynamic compaction.

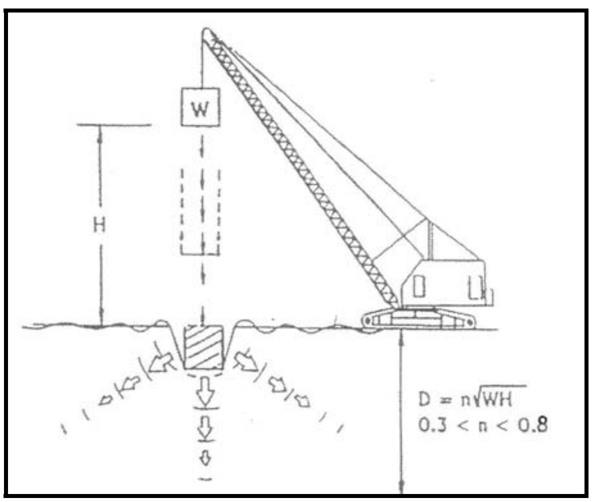


Figure 19-15, Dynamic Compaction Schematic (Ground Improvement Methods – August 2006)

19.6.1 Analysis

Dynamic compaction is used to densify natural and fill deposits to improve the soil properties and performance of the subgrade soils. The primary uses of dynamic compaction are:

- Densification of loose deposits
- Collapse of large voids
- Related applications

Dynamic compaction is used to densify loose deposits of soil by reducing the void ratio. This ground improvement method is used for pervious, granular soils (Zone 1 - sands, gravels and non-plastic silts) that meet the gradation, permeability (hydraulic conductivity) and plasticity shown in Figure 19-16. For saturated Zone 1 soils, the induced excess pore pressures from dynamic compaction cause the soil particles to lose point-to-point contact (i.e. liquefy). Following dissipation of these excess pore pressures, the soil grains settle into a more dense structure. Besides permeability, the degree of saturation, length of the drainage path, and the soil stratigraphy also affect the effectiveness of dynamic compaction. The degree of saturation is related to the position of the groundwater table. For soils located above the groundwater table, the results of dynamic compaction are immediate, while time is required to allow pore pressure dissipation of soils below the water table. Dense or hard layers near the ground surface can limit the effect of dynamic compaction on deeper soils.

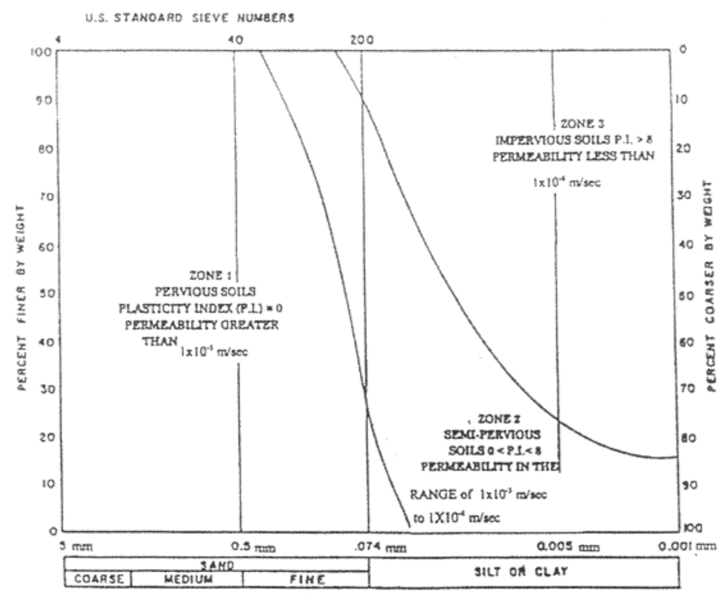


Figure 19-16, Soil Grouping for Dynamic Compaction (Ground Improvement Methods – August 2006)

Using a phase diagram, the results of multiple dynamic compaction passes verify the reduction in void ratio and the resulting densification of the subgrade soils (see Figure 19-17). It should be noted that while the void ratio decreases, the volume of the solids does not change.

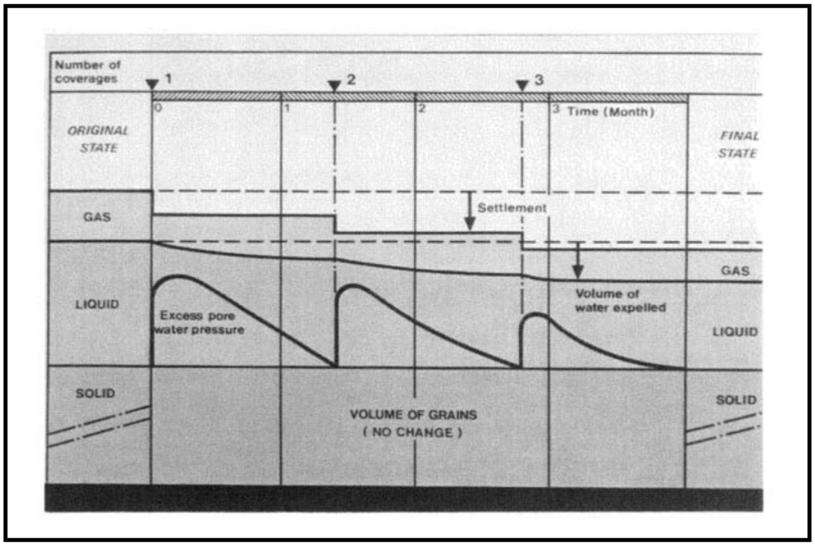


Figure 19-17, Dynamic Compaction Phase Diagram (Ground Improvement Methods – August 2006)

The soils indicated in Zone 3 (Figure 19-16) are typically impervious, plastic, fine-grained soils. The use of dynamic compaction is not recommended for these soils. The soils located in Zone 2 may be improved using dynamic compaction; however, multiple passes of the tamper will be required. In addition, additional time will be required between each pass to allow for the dissipation of excess pore pressures.

Large voids in natural or fill deposits can be collapsed using dynamic compaction depending on the depth to the void and the weight and drop of the tamper. Dynamic compaction can be used to improve fill materials of unknown compactive effort. In addition, dynamic compaction is also used to compact construction debris and solid waste materials that may be located within the Right-of-Way. Using dynamic compaction on construction debris and solid waste materials will improve the density of the material and may result in not having to remove and properly dispose of these materials.

According to Ground Improvement Methods, "In weak saturated soils relatively deep craters (> 5 feet) can develop. If these craters are filled with coarse granular materials and supplemental energy applied, the granular material will be driven into the weak deposit. This type of improvement is strictly speaking not dynamic compaction and is called dynamic replacement. The dynamic compaction equipment is used to produce the improvement, so this procedure is a related form of ground improvement. The depth of improvement is generally less than about 10 to 13 feet."

19.6.1.1 Advantages

Dynamic compaction has many advantages which are listed below:

- The tamper can be used as a probing, as well as a correcting, tool. Dropping the tamper can identify areas of loose soil or voids (deeper crater). This identification allows real time adjustments to the dynamic compaction program.

- Densification of soils can be observed as compaction proceeds. After several passes, the depth of the craters should become shallower indicating densification of the underlying soils.
- Dynamic compaction can be used on sites that have heterogeneous deposits (i.e., boulders, loose fills, construction debris, and solid waste).
- Dynamic compaction results in a bearing stratum that is more uniform after compaction, resulting in uniform compressibility, minimizing differential settlements.
- Densification can be achieved below the water table, eliminating costly dewatering.
- Standard construction equipment can be used for dynamic compaction with the exception of very heavy tampers and high drop heights. Very heavy tampers and high drop heights will require specialty contractors.
- Dynamic compaction can be performed in inclement weather, provided precautions are taken to avoid water accumulation in the craters.

19.6.1.2 Disadvantages

Dynamic compaction has the following disadvantages:

- Ground vibrations induced by dynamic compaction can travel significant distances from the point of impact, thus limiting the use of dynamic compaction to light weight tampers and low drop heights in urban environments.
- The groundwater table should be more than 6.5 feet below the existing ground surface to prevent softening of the surface soils and to limit the potential of the tamper sticking in the soft ground.
- A working platform may be required above very loose deposits. The working platform also functions to reduce the penetration of the tamper. The cost of the working platform can add significant costs to the project.
- Large lateral displacements (1 to 3 inches) have been measured at distances of 20 feet from the point of impact by tampers weighing 33 to 66 kips. Any buried structures or utilities within this zone of influence could be damaged or displaced.

19.6.1.3 Environmental Considerations

As indicated previously the vibrations created by dynamic compaction can have an adverse effect on adjoining properties. According to <u>Ground Improvement Methods</u>, "The U.S. Bureau of Mines has found that building damage is related to particle velocity. Figure *19-18* was developed by the Bureau based on experiences with damage measurements made in residential construction from blast-induced vibrations. The limiting particle velocity depends upon the frequency of the wave form. Normally, dynamic compaction results in frequencies of 5 to 12 *Hertz* (Hz). Using Figure *19-18* as a guide, this would limit peak particle velocities to values of ½-inch per second for older residences with plaster walls and ¾ inches per second for more modern constructions with drywall. Peak particle velocities that exceed the values given in Figure *19-18* do not mean damage will occur. Rather, these values are the lower threshold beyond which cracking of plaster or drywall may occur.

"Data generated by the U.S. Bureau of Mines indicate that minor damage occurs when the particle velocity exceeds 2 inches per second (51 mm/sec), and major damage occurs when the particle velocity exceeds about 7-1/2 inches per second (190-1/2mm/sec). Thus, keeping the particle velocity less than about ½ to ¾ inches per second should be a reasonably conservative value to minimize damage."

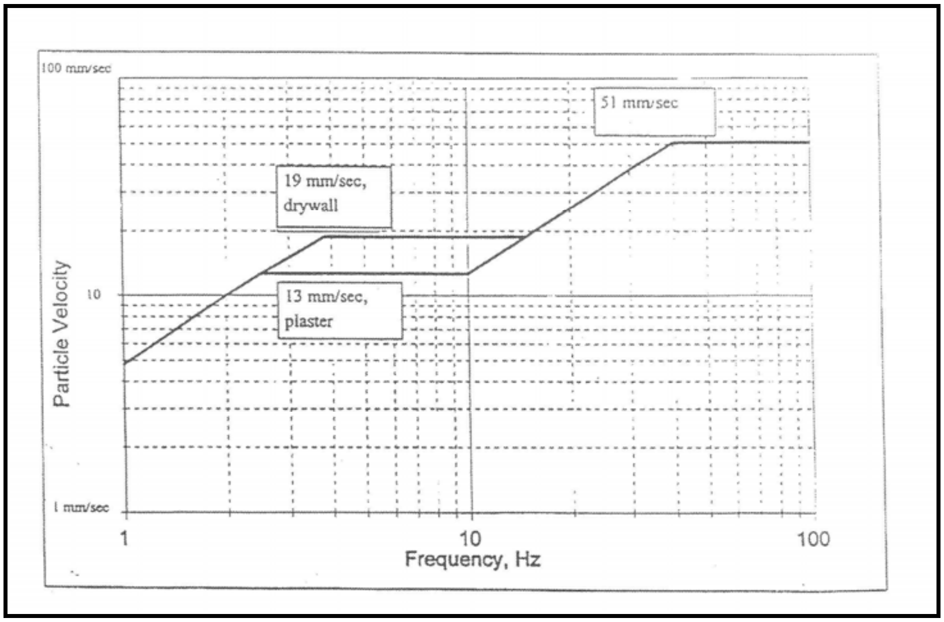


Figure 19-18, Safe Level of Vibrations for Houses (Ground Improvement Methods – August 2006)

Seismographs are typically used to measure ground velocities caused by dynamic compaction. Typically, a base line reading is obtained prior to commencing operations to obtain the level of ambient background vibrations. The readings during production operations are obtained from seismographs on adjacent structures or at the construction limits. However, prior to dynamic compaction production operations, an estimate of the particle velocity to be generated is required. Figure 19-19 can be used for planning purposes.

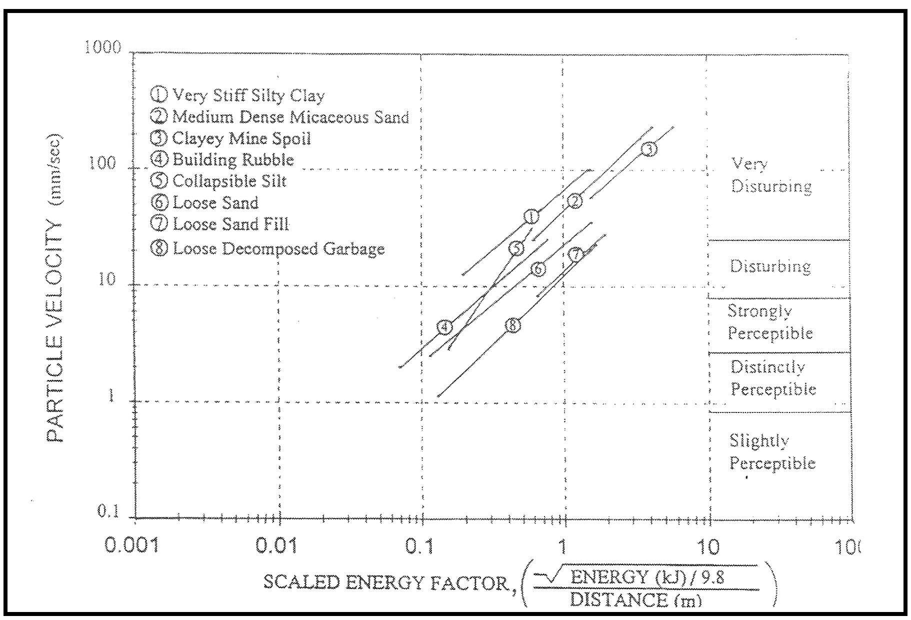


Figure 19-19, Scaled Energy Factor vs Particle Velocity (Ground Improvement Methods – August 2006)

kJ = 737.5 ft-pounds; m = .3048 ft; mm = 0.0394 inches

If the estimated particle velocity exceeds the project requirements, then, either the weight of the tamper is reduced or the drop height is lowered. Ground vibrations on the order of ½ to ¾ inches per second are perceptible to humans. Even though these vibrations should not cause damage, vibrations of this magnitude can lead to complaints. Educating the adjacent property owners to the potential impacts of the ground vibrations should be performed.

Dynamic compaction can lead to lateral soil movement. Measurements and observations from other projects has indicated tampers ranging from 33 to 66 kips should not be used within 20 to 30 feet of any buried structure, if movements can cause damage to the structure. In addition, flying debris can occur following impact of the tamper. To avoid flying debris, a safe working distance should be established from the point of impact. Dynamic compaction has an effective depth limitation of approximately 36 feet.

19.6.2 Design

After determining if dynamic compaction is a viable ground improvement method, the next step is to develop a more specific ground improvement plan including the following:

- Determining the project performance requirements for the completed structure.
- Selecting the tamper mass (weight) and drop height to correspond to the required depth of improvement.
- Estimating the degree of improvement that will result from dynamic compaction.
- Determining the applied energy to be used over the project site to produce the improvement.

19.6.2.1 Performance Requirements

Dynamic compaction densifies in-situ soils and thus improves the shear strength and reduces the compressibility of the in-situ soils. A baseline of in-situ properties should be established prior to commencing ground improvement using SPT or CPT methods. The approximate required level of improvement should be determined for the specific baseline testing procedure. Verification testing shall be conducted during the dynamic compaction operations to determine if the required amount of densification is being achieved.

19.6.2.2 Depth of Improvement

The depth of improvement is based on a number of variables including weight (mass) of the tamper, drop height, soil type, and average applied energy. The maximum depth of improvement is determined from the following equation.

$$D_{max} = n\sqrt{WH} \qquad \text{Equation 19-13}$$

Where,

D_{max} = Maximum depth of improvement (meters) (1 m = 0.3048 ft)
n = Empirical coefficient ranging from 0.3 to 0.8, but normally used as 0.5 for most soils and 0.4 is used for landfills
W = Mass of tamper (metric tonnes) (1 metric tonne = 2,205 pounds)
H = Drop height (meters)

The depth of improvement is also affected by the presence of soft or hard layers. Both types of layers absorb the energy imparted by the tamper and can therefore reduce the depth of improvement.

19.6.2.3 Degree of Improvement

As indicated above, the degree of improvement is typically measured using either SPT or CPT measurements. SPT or CPT tests are performed prior to and after dynamic compaction to monitor the amount of improvement imparted on the soil. Figure 19-20 provides a general indication of the amount of improvement from dynamic compaction.

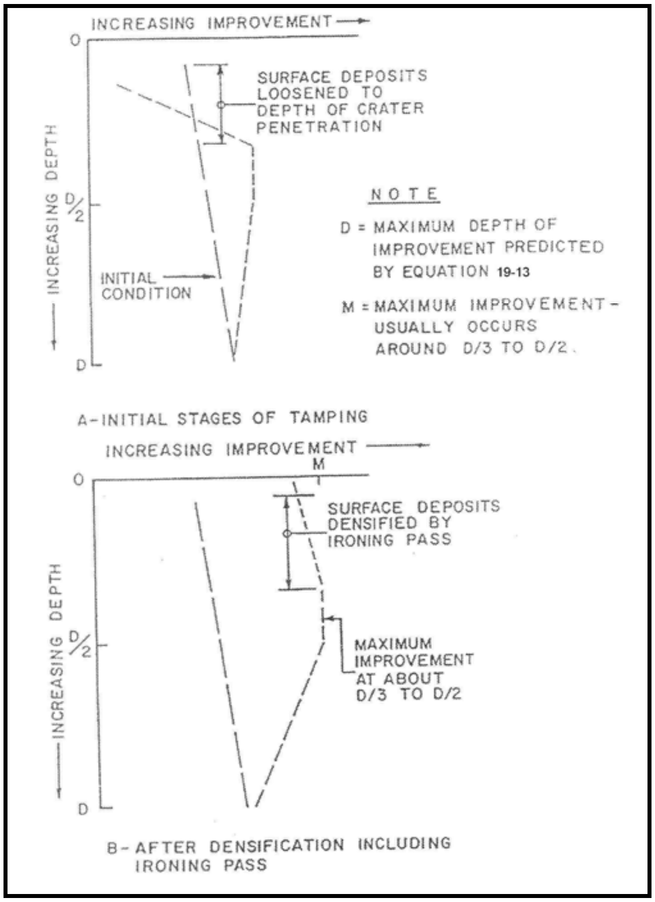


Figure 19-20, Dynamic Compaction Improvements vs. Depth (Ground Improvement Methods – August 2006)

The degree of improvement achieved is primarily a function of the average energy applied at the ground surface. Generally, the greater the amount of energy, the greater the degree of improvement; however, there are limitations to the maximum SPT or CPT values that can be achieved. These maximum values are listed in Table 19-12. These maximum values occur at improvement depth ranges of D/3 to D/2, above or below this range the test values would be less. These maximum values should only be used as a guide. The actual degree of improvement should be determined during and after the completion of dynamic compaction. The degree of improvement can continue to increase for months or, in some cases, years following the complete dissipation of excess pore pressures.

Table 19-12, Upper Bound Test Values after Dynamic Compaction (Ground Improvement Methods – August 2006)

Soil Type	Maximum Test Values	
	N-values (bpf)	Cone Tip Resistance (tsf)
Sand & Gravel	30 – 50	200 – 300
Sandy Silts	25 – 35	135 – 175
Silts & Clayey Silts	20 – 35	105 - 135
Clay fill & Mine spoil	20 – 40[1]	N/A
Landfills	15 – 40[1]	N/A

[1]Higher test values may occur because of large particles in the soil mass.

19.6.2.4 Energy Requirements

According to Ground Improvement Methods, “Dynamic compaction is generally undertaken in a grid pattern throughout the area. For this reason, it is convenient to express the applied energy in terms of average values. This average applied energy can be calculated on the basis of the following formula:”

$$AE = \frac{(W)(H)(N)(P)}{(G)^2} \quad \text{Equation 19-14}$$

Where,

AE = Applied energy
N = Number of drops at each specific drop point location
W = Tamper weight
H = Drop height
P = Number of passes
G = Grid spacing

The average applied energy is the sum of all different size tampers and drop heights. Normally, high energy is achieved using a heavy tamper dropped from a high height. This is frequently followed by the ironing pass (low level energy). The ironing pass is conducted using smaller sized tampers being dropped from lower heights. For planning purposes, the estimated required energy can be obtained from Table 19-13.

Table 19-13, Applied Energy Guidelines (Ground Improvement Methods – August 2006)

Soil Deposit	Unit Applied Energy ($ft\text{-}lb/ft^2$)	Percent Standard Proctor Energy[1]
Zone 1 Soils[2]	4,130 – 5,170	33 - 41
Zones 2 and 3[2]	5,170 – 7,230	41 - 60
Landfills	12,400 – 22,700	100 - 180

[1]Standard Proctor energy equals 12,400 $ft\text{-}lb/ft^2$

[2]Refer to Figure19-16

19.7 DEEP SOIL MIXING

Deep soil mixing is a ground improvement technique that mixes reagents into the soil at a specific depth to improve the in-situ soil properties without requiring excavation or removal. Deep soil mixing mixes the soil and reagent together, whereas grouting injects cementitious materials into the in-situ soil matrix to improve the soil. Grouting is discussed below. Deep soil mixing can be used for a variety of applications including excavation support, soil stabilization, settlement reduction, foundation support, and mitigation of liquefaction potential. Deep soil mixing is performed under many proprietary names, acronyms and processes worldwide. However, the basic concepts and procedures are similar for all techniques. The mixed soil product and the objectives of the mixing program can be divided into standard generic terms as presented in the table below:

Table 19-14, Deep Soil Mixing Generic Terms
(Ground Improvement Methods – August 2006)

Method of reagent injection	Wet (**W**) or Dry (**D**)
Method of reagent mixing	Rotary energy (**R**) or High-pressure jet (**J**)
Location of mixing action	End of drilling tool (**E**) or Along shaft (**S**)

These generic terms can be combined into four distinct processes of deep soil mixing (see Figure 19-21), WRS, WRE, WJE and DRE. Some of the possible combinations of deep soil mixing methods do not exist. For example DJE (dry, jet end) does not exist. Jetting is a wet method and, therefore, could not be used with a dry mix application.

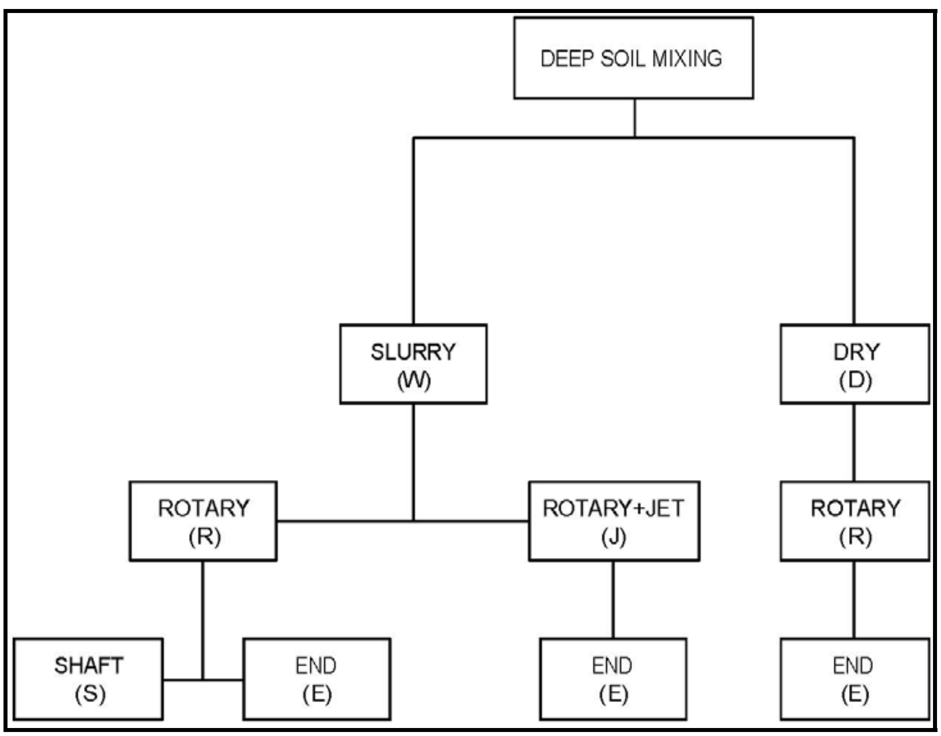


Figure 19-21, Generic Classification of Deep Soil Mixing Techniques
(Ground Improvement Methods – August 2006)

The four processes discussed previously can be divided into two groups as indicated in Table 19-15.

Table 19-15, Deep Soil Mixing Groups
(Ground Improvement Methods – August 2006)

Wet Deep Soil Mixing Methods	WRS, WRE, WJS	Refers to wet, single or multi-auger, block or wall developed for large-scale foundation improvement in any soil. Primary reagents are cement-based.
Dry Deep Mixing Methods	DRE	Refers to dry, single-auger column technique developed for soil stabilization and reinforcement of cohesive soils. Primary reagents are granular or powdered lime for lime columns and cement or lime-cement mixtures.

19.7.1 Analysis

Wet, deep soil mixing methods are typically used for large-scale structural support improvement, while dry deep soil mixing methods are used primarily for soil stabilization/reinforcement and settlement reduction. Discussed in the following paragraphs are applications, of wet and dry deep soil mixing that are typical for transportation related projects. For other applications see Ground Improvement Methods.

Wet deep soil mixing methods have been used to stabilize soil to provide an improved foundation bearing capacity and for seismic stabilization. The most common usage is for settlement control and/or shear strength improvement under embankments. Under this usage wet deep soil mixed columns are constructed in grid or lattice geometry to provide additional resistance to bending. This same method can be used to improve the mass shear strength of a potentially liquefiable soil as well as contain liquefaction propagation.

Dry deep soil mixing methods such as lime, cement, or lime-cement columns have been used to improve soft, cohesive soils. Lime-cement columns have been used to reduce total and differential settlements using rationale similar to stone columns. These columns are stiffer and relatively less compressible than the surrounding soil; therefore, carry a greater portion of the applied load thus reducing total and differential settlement. The amount of settlement reduction is a function of the area replacement ratio and the stress concentration ratio, which is a function of the column stiffness compared to the untreated soil. These types of columns are used to reinforce existing soils by increasing the mass shear strength, thus increasing the stability of embankments and slopes. Typically, the columns are placed in a grid pattern under the embankments and in interconnected rows under the slope to provide sufficient resistance to bending. Lime, cement, or lime-cement columns can be used to increase the stability of anchored sheet pile walls. The columns increase the passive earth pressure at the toe of the wall. In addition, columns placed behind the wall can reduce the lateral earth pressure acting on the sheet piles.

19.7.2 Advantages and Disadvantages/Limitations

19.7.2.1 Wet Deep Soil Mixing Methods

The advantages of wet deep soil mixing are, it can be performed to depths up to 100 feet and can, conceptually, be used for most subsurface conditions, from soft, plastic clays to medium dense sands and gravels with cobbles. However, this method is primarily used to improve soft cohesive and loose to medium dense cohesionless soils. Deep soil mixing uses the in-situ soil, making this method more economical than removal and replacement. The problems associated with disposal of the waste material are considerably reduced in an amount proportional to the percentage of additives used and the moisture content of the in-situ soils. The construction is a drilling process which is ideal in noise and vibration sensitive areas.

The disadvantages/limitations of wet deep soil mixing are the relative high cost of mobilization of the mixing equipment plus the cost of accompanying auxiliary batch plants. Wet deep soil mixing is uneconomical for small projects. A more extensive geotechnical exploration is required prior to using wet deep soil mixing than is typical. In addition, bench scale testing must be conducted and may require several months to complete. Dense cohesionless soils can not be readily penetrated by the existing deep soil mixing equipment. The amount of spoil produced by deep soil mixing is generally less than for some ground improvement methods. Spoil generation can range from thirty to one-hundred percent depending on project specifics, equipment and methods used, and in-situ moisture content. Disposal of this spoil can add significant cost to a project. There is a lack of well developed design and analysis models available. Lastly, there is no standardized method of quality control testing, making design verification difficult and subjective.

19.7.2.2 Dry Deep Soil Mixing Methods

One advantage of dry deep soil mixing methods in soft clay is that it often provides an economic benefit when compared to other conventional foundation methods. This advantage is based on several project factors including size, weight, and flexibility of the structure, depth, and shear strength of the compressible layer, the risks, and consequences of failure and the effects of lowering the groundwater table. Using lime or lime-cement columns can reduce the consolidation time required beneath a roadway embankment by increasing the permeability or stiffness of the columns. Another advantage to the dry deep soil mixing method is little to no spoil is generated by this method, thus eliminating the high cost of spoil disposal.

One of the disadvantages/limitations of dry deep soil mixing methods is the full strength of the columns may not be mobilized when the pH of the groundwater is acidic or the content of carbon dioxide (CO_2) is high. Low strength development should also be anticipated when mixing non-reactive cohesive soils (clays lacking pozzolans). The air-driven injection process may accumulate large quantities of air in the ground potentially causing heave of the adjacent ground surface. This problem can be eliminated by adding mixing paddles to the mixing tool and/or substantially increasing the mixing time. The creep strength of the columns and the shear strength of the stabilized soil is time dependent. Therefore, several months may be required to perform the laboratory bench scale testing. The average shear strength of the stabilized soil has to be at least three to five times the initial shear strength before dry deep soil mixing becomes economical. There is a lack of well developed design and analysis models available. Lastly, there is no standardized method of quality control testing, making design verification difficult and subjective.

19.7.3 <u>Feasibility</u>

The feasibility of using deep soil mixing shall be determined prior to recommending this ground improvement method. The feasibility evaluation includes, but is not limited to, a site investigation, a feasibility assessment, and preliminary testing (bench scale testing).

19.7.3.1 Site Investigation

The site investigation required for deep soil mixing exceeds the requirements contained in this Manual. If deep soil mixing is selected as an alternate ground improvement method, then, additional site specific information will be required. The proposed site investigation plan shall be submitted to the GDS and the PCS/GDS for concurrence prior to execution. Prior to commencing the site investigation, observations of the proposed construction area should be made to include ground surface condition, the presence of overhead or underground utilities, site access, and any other observations that could affect the ability to use this method. It should be noted that typically the equipment used for deep soil mixing is relatively large and will require more space to operate in. In addition, use of the wet methods may generate large amounts of spoil, and it should be determined if there is adequate space on site to store this material. The site investigation should include the following items:

- Evaluation of the subsurface: predominant soil type; existence of any obstructions; existence and percentage of organic matter
- Natural moisture content
- Engineering properties: strength and compressibility
- Classification properties: moisture-plasticity relationship and grain-size distribution
- Chemical and mineralogical properties to include assessment for the presence of pozzolanic materials, including soluble silica and alumina, which can affect lime reactivity only
- Ground water levels

19.7.3.2 Assessment

Deep soil mixing is best used when the subsurface conditions are soft to loose with no obstructions to depths no greater than 100 feet. There should be unrestricted overhead clearance and a need for relatively vibration free ground improvement methods. Deep soil mixing will cause the temporary loss of in-situ soil strength, which may affect adjacent structures. The assessment should review the information obtained from the site investigation. Selected soil chemical properties are provided in the table below.

Table 19-16, Favorable Soil-Chemistry Factors
(Ground Improvement Methods – August 2006)

Property	Favorable Soil Chemistry
pH	> 5
Natural moisture content	< 200 (dry method) < 60 (wet method)
Organic content	< 65 (wet method)
Loss on Ignition	< 10
Humus Content	< 1
Electrical conductivity	0.4 mΩ/cm

19.7.3.3 Preliminary Testing

After assessing the viability of soil for deep soil mixing, samples should be prepared to determine the water, soil, reagent ratios as well as determining the time required for mixing.

The samples should then be tested for unconfined compressive strength at various curing times to determine strength gains with time. This entire process can be called preliminary or bench scale testing. The preliminary testing results will assist in narrowing the potential improvements levels that can be achieved in the field. These results should be compared to the typical results presented in the table below. It is important to note that very important variables associated with equipment mixing capabilities, such as rate of penetration and withdrawal, mixing energy, and vertical circulation of materials, cannot be modeled by the laboratory testing program.

Table 19-17, Typical Improved Engineering Properties (Ground Improvement Methods – August 2006)

Property	Typical Range
Unconfined Compressive Strength, q_u	Cohesionless Soils – 29 – 725 psi Cohesive Soils – 29 – 435 psi
Hydraulic Conductivity, k	10^{-4} – 10^{-7} cm/s
Young's Modulus (E_{50}) [Secant Modulus at 50% q_u]	100 – 300 q_u
Tensile Strength (wet mix)	8 – 14 percent of q_u
Poisson's Ratio	0.19 – 0.45 typically 0.26

Provided in the table below are guidelines related to the penetration, mixing speed, water cement ratio, and reagent content typically used in practice.

Table 19-18, Mixing Guidelines (Ground Improvement Methods – August 2006)

Reagent Content	9-1/2 – 22-1/2 pcf
Mixing Rotational Speed	20 – 45 rpm
Penetration Rate	1 yd/min
Water Cement Ratio	0.6 – 1.3 but 1.0 is normal

According to Ground Improvement Methods, "Much recent research and interest has been directed toward developing indicators of the potential efficiency of the mixing process that would produce a more homogeneous in-situ product of higher strength. It has been suggested by Japanese researchers that efficiency of a particular system can be established or expressed in terms of 'the number of mixing per *yard*, T,' which is related to certain operational and reagent injection characteristics as follows:"

$$\boldsymbol{T} = \boldsymbol{N}\left[\left(\frac{\boldsymbol{R_p}}{\boldsymbol{S_p}} \bullet \frac{\boldsymbol{W_i}}{\boldsymbol{W}}\right) + \frac{\boldsymbol{R_w}}{\boldsymbol{S_w}}\right] \qquad \text{Equation 19-15}$$

Where,

N = Total number of mixing blades
S_p, S_w = Penetration and Withdrawal speed (yard/min)
R_P, R_w = Blade rotation speed during penetration and withdrawal (rpm)
W_i = Stabilizer (reagent) injection on penetration (pcf)
W = Total amount of stabilizer (reagent) (pcf)

T should be greater than 350 for clays and range from 400 to 450 for peaty soils according to the research to develop a good quality product.

19.7.4 Design

Deep soil mixed columns are designed similarly to stone columns in that unit cell concepts, stress ratios (n) and area replacement ratios (α_s) are used for design. For settlement reduction, area replacement ratios on the order of 0.2 to 0.3 are used for triangular or square column patterns. Determining the strength to support the embankment load, maximizing the benefit of arching between columns, and providing the required global shear strength to ensure stability develops the optimum design spacing. Larger area replacement ratios are indicative of more stringent settlement criteria. Large area deep soil mixing columns can be used to support structures provided stability (bearing, sliding and overturning) and performance (total and differential settlement) are satisfied. Deep soil mixing columns can also be used to mitigate the potential for liquefaction by either confining the materials that will liquefy or by increasing the Cyclic Resistance Ratio (CRR) (see Chapter 13) through increasing the shear strength of the soil.

The area replacement ratio (α_s) is defined as:

$$\alpha_s = \frac{A_s}{A} \qquad \text{Equation 19-16}$$

Where,

α_s = Area replacement ratio
A_s = Area of the soil mixed column
A = Total area within the unit cell

The transfer of the applied load to the soil mixed columns from the in-situ soils depends on the relative stiffness of the soil mixed columns to the in-situ soils as well as the spacing and diameter of the soil mixed columns. Because the soil mixed columns and the in-situ soils deflect (strain) approximately equally, the soil mixed columns must be carrying a greater portion of the load (stress) than the in-situ soils. This concept has also been called the equal strain assumption. This concept has been proven by both field measurements as well as finite element analysis. The relationship between the stress in the stone column and the stress in the in-situ soil is defined in the following equation:

$$n = \frac{\sigma_s}{\sigma_c} \qquad \text{Equation 19-17}$$

Where,

n = Stress ratio or stress concentration
σ_s = Stress in the soil mixed column
σ_c = Stress in the surrounding soil

Equilibrium of vertical forces for a given α_s is provided by the following equation.

$$q = \sigma_s \alpha_s + \sigma_c (1 - \alpha_s)$$ Equation 19-18

Where,

q = Average stress on the unit cell

The stresses in the soil mixed column and the surrounding soil in the unit cell can be determined by rearranging the above equation.

$$\sigma_c = \frac{q}{[1 + (n-1)\alpha_s]}$$ Equation 19-19

$$\sigma_s = \frac{nq}{[1 + (n-1)\alpha_s]}$$ Equation 19-20

According to Ground Improvement Methods, "The total undrained shear resistance τ of the stabilized soil is assumed to correspond to the sum of the shear strengths of the column and the soil between the columns and can be evaluated from"

$$\tau = \tau_f \alpha_s + C_u (1 - \alpha_s)$$ Equation 19-21

Where,

τ_f = Undrained shear strength of soil mixed column

C_u = Undrained shear strength of soil between columns

α_s = Area replacement ratio

"Typical area *replacement* ratios are on the order of 0.20 to 0.40 and are varied until the targeted minimum total undrained shear resistance of the stabilized soil is calculated. It is anticipated that area replacement ratios of 0.20 to 0.33, and stress ratios of between 4 and 6, would *be* used typically for either block- or column-type patterns *(see Figure 19-22)*. The reduction in settlement is attributed to the concept that *the soil mixed* columns that are stiffer than the adjoining soil will carry more load."

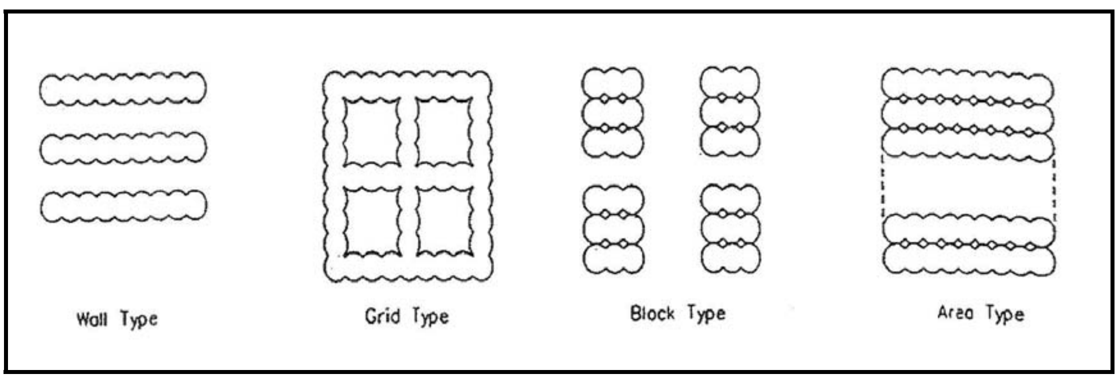


Figure 19-22, Deep Soil Mixing Treatment Patterns (Ground Improvement Methods – August 2006)

19.7.5 Wet Soil Mix Material Properties

The properties of wet soil mixing are influenced by the soil type and chemistry, in-situ water content, amount of reagent used, water-reagent ratio of slurry, degree of mixing, curing environment, construction process and equipment, spoil generated, and age. The in-situ strength of the treated soil can be one-half to one-fifth of the strength measured in the laboratory. Therefore, the strength of the mixed soil prepared in the laboratory should only be used as an indication of the level of improvement that is achievable in the field. Typically, the longer a mixed soil cures, the greater the increase in strength. Field testing has shown increases in mixed soil strength up to 6 months after mixing. Provided in the following table are typical compressive shear strengths.

Table 19-19, Typical Improved Compressive Strength, Wet Mix Method (Ground Improvement Methods – August 2006)

In-Situ Soil	Improved Compressive Strength (psi)
Organic and very plastic clays	175
Soft clays	60 – 220
Medium/hard clays	100 – 360
Silts	145 - 435
Fine to medium sands	220 – 725

19.7.6 Dry Soil Mix Material Properties

Dry mix methods should be used when the in-situ soils consist of soft clays with in-situ moisture contents between sixty and one-hundred and twenty percent and where the required increase in strength is less than 145 psi. Typically, dry mix methods use either plain lime or lime-cement as the reagent. The lime-cement modified soils will have higher shear strength than the lime only stabilized soils. As with wet mixed soils, the shear strength obtained from laboratory prepared specimens should be reduced. The shear strength should be reduced by approximately one-third to one-half, but should be no greater than 8.4 ksf.

19.7.7 Verification

The properties of the improved ground require verification to ascertain whether the requirements of the project are being met. The contractor should be required to conduct laboratory (bench scale) testing to verify that proposed construction methods and mixes will achieve the requirements of the contract. After completion of the mixing, either in-situ testing or obtaining cores for laboratory testing should be performed. The in-situ testing can consist of cone penetrometer testing (CPT), dilatometer testing (DMT), standard penetration testing (SPT), or pressuremeter testing (PMT).

19.8 GROUTING

According to Ground Improvement Methods, "Grouting comprises a variety of techniques that employ injection of a range of materials into soil or rock formations, via boreholes, to alter the physical characteristics of the formation when the materials set. More specifically, grouting can be used to fill fissures and voids in rock, to fill voids between the ground and overlying structures, and to treat soils to enhance strength, density, permeability, and/or homogeneity." The type of grouting used is based on the anticipated/required results and the soil/rock that the grouting is being used in. A successful grouting program consists of a detailed geotechnical investigation, active monitoring during construction, and verification that the grouting program is meeting the project requirements.

The geotechnical investigation is more detailed than is normally performed to identify in-situ conditions that could affect the effectiveness of the grouting program. The results of this detailed investigation are used to select the type of grouting, as well as the grouting materials. In addition, the investigation will aide in determining the potential effectiveness of the grouting program. To improve effectiveness, a real time monitoring plan is required, which allows for field adjustments to the grouting program to account for changes in subsurface conditions. Finally, a comprehensive grouting program shall include a means of verifying that the required results are being achieved.

The definitions contained in the Ground Improvement Methods manual are used in this Manual. The Ground Improvement Methods manual identifies two principle types of grouting which are listed in the table below. Figure 19-23 provides schematics of the various types of grouting.

Table 19-20, Types of Grouting Method
(Ground Improvement Methods – August 2006)

Principle Type of Grouting	Specific Type of Grouting
Rock Grouting	Fissures (using High Mobility Grouts (HMG))
	Voids (natural and artificial, using Low Mobility Grouts (LMG))
Soil Grouting	Permeation (using HMG and solution grouts)
	Compaction (or displacement)
	Jet (or replacement)
	Fracture (including compensation grouting)

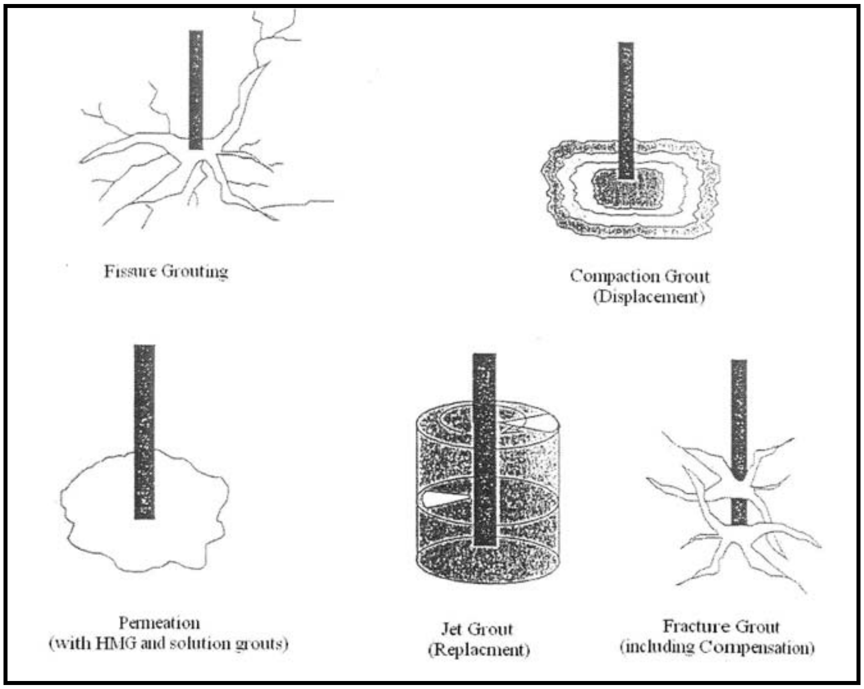


Figure 19-23, Types of Grouting Schematic
(Ground Improvement Methods – August 2006)

19.8.1 Grout Materials

There are four categories of grouting materials, which are listed below:

1. Particulate (suspension or cementitious) grout
2. Collodial solutions
3. Pure solutions
4. Miscellaneous materials

Category 1 grouts are comprised of mixtures of water and particulate solids. The particulate solids may consist of cement, fly ash, clays or sands. These mixtures are stable and have cohesion and plastic viscosity increasing with time. Due to their basic characteristics and relative economy, these grouts remain the most commonly used for both routine waterproofing and ground strengthening. The water to solids ratio is the prime determinant of their properties and basic characteristics such as stability, fluidity, viscosity, and strength durability. Neat cement or clay/bentonite-cement grouts are comprised of Portland cement or microfine cement depending on the size requirements of the grout. Figure 19-24 shows the increase in apparent viscosity with time for these grouts and Figure 19-25 shows grain-size distribution of various cements.

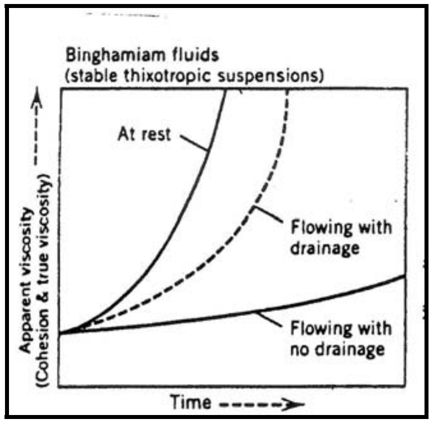


Figure 19-24, Viscosity versus Time for Category 1 Grouts (Ground Improvement Methods – August 2006)

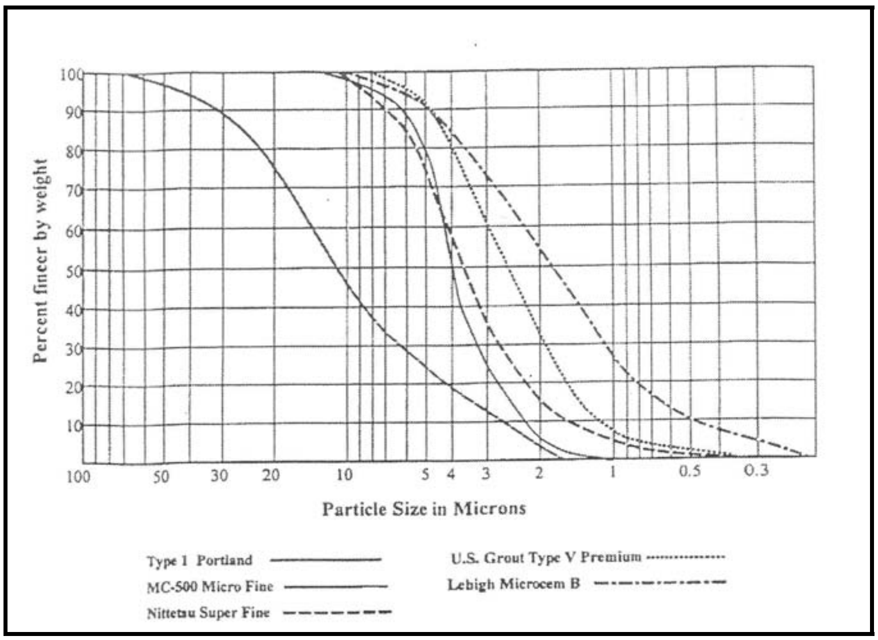


Figure 19-25, Grain-Size Distribution of Cements (Ground Improvement Methods – August 2006)

Category 2 and 3 grouts, commonly called solution or chemical grouts, are typically subdivided based on component chemistries; for example, silicate based (Category 2) (colloidal) or resin based (Category 3) (pure solution). Figure 19-26 provides an indication of the change of viscosity with time for these grouts. Category 2 grouts are colloidal solutions that are comprised of mixtures of sodium silicate and a reagent, which when mixed, change viscosity over time to a gel. Sodium silicate is an alkaline, colloidal aqueous solution, while the reagents may be organic or inorganic (mineral). The common types of organic reagents are monoesters, diesters, triesters and aldehydes. These reagents react with the sodium silicate to produce acid as a by-product and can produce either a soft or hard gel depending on the concentration of each compound. The inorganic reagents contain cations that are capable of neutralizing the silicate alkalinity. Typical inorganic reagents are sodium bicarbonate and sodium aluminate. The relative proportions of silicate and reagent will be determined by their own chemistry and concentration, the desired short- and long-term properties, such as gel setting time, viscosity, strength, syneresis and durability, as well as cost and environment acceptability.

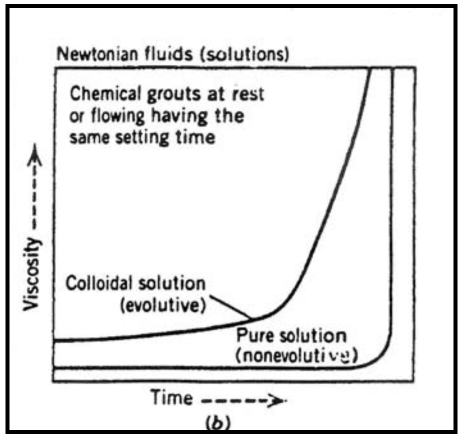


Figure 19-26, Viscosity versus Time for Category 2 and 3 Grouts (Ground Improvement Methods – August 2006)

Category 3 grouts are known as pure solutions since these grouts consist of resins. The resins are solutions of organic products in water or a nonaqueous solvent that are capable of causing the formation of a gel with specific mechanical properties under normal temperature conditions and in a closed environment. These grouts exist in the following forms, characterized by the mode of reaction or hardening:

- Polymerization – Activated by the addition of a catalyzing agent (polyacrylamide resins)
- Polymerization and Polycondensation – Arising from the combination of two components (epoxies or aminoplasts)

The setting times for these grouts is adjusted by varying the proportions of the reagents or components. According to Ground Improvement Methods, "Resins are used when particulate grouts or colloidal solutions prove inadequate, for example when the following grout properties are needed:

- Particularly low viscosity
- Very fast gain in strength (a few hours)
- Variable setting time (few seconds to several hours)
- Superior chemical resistance
- Special rheological (psuedoplastic)
- Resistance to high groundwater flows"

In applications where the durability of the grout is important, resins are typically used for both strength and waterproofing. Resins may be divided into four subcategories as indicated in Table 19-21.

Table 19-21, Types, Use, and Applications of Resins
(Ground Improvement Methods – August 2006)

Type of Resin	Applicable Ground Type	Use/Application
Acrylic	Granular, very fine soils Finely fissured rock	Waterproofing by mass treatment Gas tightening (mines, storage) Strengthening up to ~15 tsf Strengthening of a granular medium subjected to vibrations
Phenol	Granular, very fine soils	Strengthening
Aminoplastic	Schists and coals	Strengthening (by adherence to materials of organic origin)
Polyurethane	Large Voids	Formation of a foam that forms a barrier against running water (using water-reactive resins) Stabilization or localized filling (using two-component resins)

There are only two types of polyurethanes that are appropriate for grouting. These types are listed in Table 19-22.

Table 19-22, Polyurethane Types
(Ground Improvement Methods – August 2006)

Polyurethane Type	Properties
Water Reactive	Liquid resin reacts with groundwater to form either flexible (elastomeric) or rigid foam These resins take two forms: • Hydrophobic – react with water, but repel it after the final (cured) product has formed • Hydrophilic – react with water, but continue to physically absorb it after the chemical reaction has been completed
Two Component	Two compounds in liquid form react to provide either a rigid foam or an elastic

Category 4 grouts (Miscellaneous grouts) are composed of organic compounds or resins. These grouts are used primarily for strengthening and waterproofing, but may also have very specific qualities such as resistance to erosion or corrosion, and flexibility. The use of Category 4 grouts may be limited by specific concerns such as toxicity, injection, handling difficulties, and cost. In addition, many of these grouts are proprietary in nature, which can make their use difficult at best. Category 4 grouts are composed of hot melts, latex, polyesters, epoxies, furanic resins, silicones, and silacols. Some of these types have limited use in ground improvement. Category 4 grouts should only be used if there are either no other options or if the grouting system (grout and application of the grout) is fully understood by both the designer and the contractor.

19.8.2 Rock Grouting

There are two types of rock grouting: rock fissure grouting and void filling. Both types of grouting are discussed briefly in the following sections.

19.8.2.1 Rock Fissure Grouting

The grouting of rock fissures is primarily used to provide hydraulic cut-offs and has the added benefit of binding the rock mass together thus improving the load bearing capability. Rock fissure grouting typically has limited applications on transportation projects. However, rock fissure grouting can be used to stabilize rock slopes, remediate road tunnels, repair drilled shafts, and seal of drilled shaft boreholes from the in-flow of ground water. The variability of the rock mass can make this ground improvement technique extremely difficult to predict the results of using the method. Because of the variability in the rock mass, often a design phase test program is conducted to determine the effectiveness of the rock fissure grouting program. Using the results of the test program, the final design can be completed and a program cost can be estimated.

The use of rock fissure grouting has the advantage of being less expensive when compared to other repair options of weak rock, such as removal, replacement, or abandoning the site. However, the actual cost of rock fissure grouting can vary considerably because of potential variation of the rock mass within the site boundaries. Further, poor field practices can lead to unsatisfactory performance of the rock grouting. These include inducing uplift that results from excessive pressures, premature plugging of fissures, unsuitable injection methods or formulations or by inappropriate drilling and flushing methods and improper hole spacing or improper orientation of the grout holes.

The primary purpose of this form of rock grouting is the sealing of cracks and fissures within the rock mass. The main consideration in rock grouting is the grain-size of the particulate grout compared to the width of the rock fracture to be grouted.

$$\boldsymbol{N_R = \frac{f_w}{D_{95}}} \qquad \text{Equation 19-22}$$

Where,

N_R = Groutability ratio of rock
f_w = Fissure width
D_{95} = Grout diameter at 95 percent finer
N_R > 5 – Grouting consistently possible
N_R < 2 – Grouting not possible

While the fissure width cannot be changed, the fineness of the grout can be controlled, thus producing a groutability ratio that can be increased to greater than two. Rock grouting with particulate materials normally falls into one of the categories indicated in Table 19-23.

Table 19-23, Rock Grouting Categories
(Ground Improvement Methods – August 2006)

Rock Grouting Category	Description
Curtain	Drilling and grouting of two or more lines of grout holes to an impermeable material to produce a barrier to seepage.
Area	Grouting a shallow zone in a particular area by utilizing grout holes arranged in a pattern or grid to mechanically improve fractured or jointed rock.
Tunnel	Used to fill voids behind tunnel liners, treatment of material surrounding the bore or seepage control. Pre-excavation grouting from the surface or the face may be required for ground strengthening and water control.
Backfilling	Filling subsurface exploration boreholes and grout holes is important to maximize structural stability, to control water, or to prevent passage of contaminants to underlying strata.

19.8.2.2 Rock Void Grouting

Rock void grouting is used to fill natural (karstic limestone features or salt solution cavities) voids or man-made (mining activities) voids. Typically, neither of these features occurs in South Carolina. However, there are some localized areas of karstic limestone features caused by localized dewatering for mining activities. Ground Improvement Methods includes slabjacking (mudjacking) as a subset of rock void grouting. Slabjacking is the process of injecting grout under pressure to raise and relevel concrete paving (typically bridge approach slabs) that have settled (see Figure 19-27).

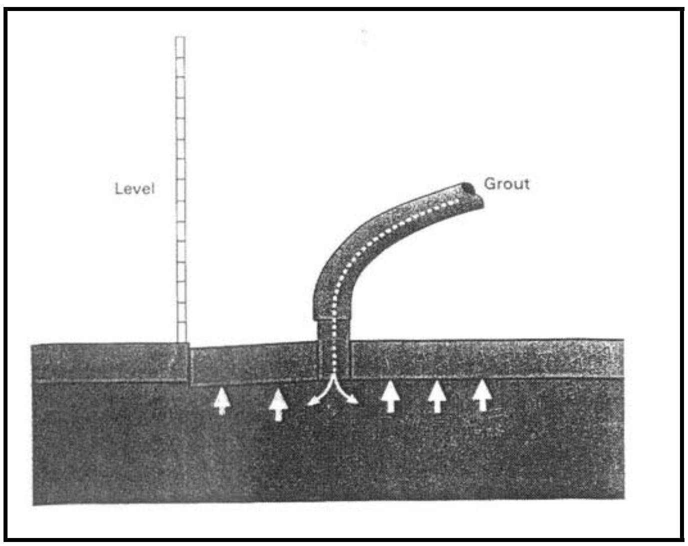


Figure 19-27, Slabjacking Schematic
(Ground Improvement Methods – August 2006)

Rock void grouting can also be used for the remediation of some scour issues. However, it will not be discussed in this Manual. Contact the PCS/GDS for guidance in the use of this method for remediation of scour. As indicated previously, slabjacking is used to correct the settlement of concrete slabs placed over compressible soils or to replace soils that have eroded away from beneath the slab. Typically, this method is used to correct problems associated with the vertical displacement of bridge approach slabs. According to Ground Improvement Methods, "Slabjacking procedures include raising or leveling, under-slab void filling (no raising), grouting slab joints, and asphalt subsealing. Most slabjacking uses a suite of cementitious grouts, incorporating bentonite, sand, ash and/or other fillers, as dictated by local preference and the project conditions and goals. Certain proprietary methods use expanding chemical foams to create uplift pressures. Best results (when no cracking is caused to the slabs) are obtained when the slabjacking is uniformly and gradually conducted. Slabjacking can also be used to "pump" *sections of rigid pavements* that have sunk below the adjoining section so that the expansion joint may be repaired are have it's functionality restored."

Slabjacking has the following advantages:

- Frequently, the most economical repair method
- Usually faster than other solutions, especially compared to removal and replacement
- Planned so that there is little disruption to the existing facility, and can be performed at times of light or no traffic
- The equipment needed to perform the slabjacking operation can be removed from the repair location, providing for maximum accessibility
- Increased load capacity of the slab is provided
- The useful life of the concrete pavement is extended
- A smoother riding surface is established

Following are the disadvantages of slabjacking:

- Cracks already present may tend to open up when the slab is treated, unless great care is taken with the process
- Slabjacking may not be cost-effective on small projects
- The original cause of the settlement is not addressed

The feasibility of using slabjacking should be based on the cost of slabjacking versus the cost of removal and replacement of the slab. Included in this evaluation should be the time required for both operations and if a roadway must be closed to perform this operation. In addition, slabjacking should not be considered when the slab is severely cracked.

After determining that slabjacking is feasible, the design should begin with understanding the underlying problem and determining the desired results of the slabjacking. If the underlying problem is settlement of soft or organic soils, then, future slabjacking may be required. Regardless of the cause of the problem, the engineer should accurately specify the required performance and tolerances for the project. Another consideration is the appearance of the finished surface. Most slabs that have settled contain some cracks. The cracks will remain visible even if the slabjacking process does not create new cracks. Further, the restored slab will also contain patches from the injection holes. The injection holes are usually on 5- to 6-foot grid spacing. The objectives of slabjacking are to fill voids and raise the slab approximately to its original elevation, without causing additional damage to the slab. Instrumentation, as simple as a string line can provide this, although the use of lasers is more accurate.

19.8.3 Soil Grouting

Soil grouting programs are used to achieve a variety of ground improvement objectives. The two main objectives of a grouting program are, first, water control and waterproofing and second, structural improvement. Waterproofing is used mainly in conjunction with new construction and water control is used mainly in conjunction with remedial applications. Structural grouting is used to improve the density of a soil, raise settled structures, control settlement, underpin, mitigate liquefaction, and control water. There are four different types of grouting that can be used on soil:

1. Permeation
2. Compaction
3. Jet
4. Soil Fracture

All four of these types of grouting can be used for water control, waterproofing and structural enhancement. These four types are discussed in greater detail in the following sections. Soil grouting has a distinct economic advantage over removal and replacement. Grouting is also generally less disruptive to the surrounding work area. Soil grouting also has some disadvantages, such as compaction grouting in fine saturated soils. Instead of squeezing the pore water out, the soil may simply displace and not consolidate or densify. Permeation grouting using certain chemical grouts may represent toxicity dangers to the groundwater and underground environment. Low toxicity chemical grouts are now available and should be specified except for unusual circumstances. Each grouting method can cause ground movement and structural distress.

The general limitation of soil grouting is the soil type to be treated. Although the range of soil grouting available encompasses most soil types, individual methods are limited to specific soils as shown in Figure 19-28.

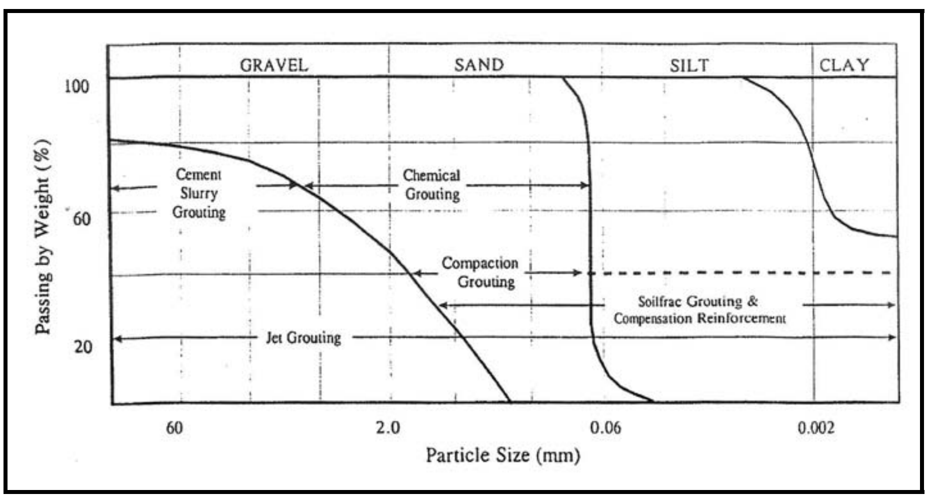


Figure 19-28, Range of Applicability of Soil Grouting Techniques (Ground Improvement Methods – August 2006)

Grouting is normally used to solve construction problems related to geological anomalies or environmental conditions. Soil grouting uses the existing soils, improving these soils, by grouting to correct deficiencies in the soil. According to Ground Improvement Methods, "Grouting of a soil involves the following sequential steps:

- Establishing specific objectives for the grouting program (designer)
- Defining the geometric and geotechnical project conditions (designer)
- Developing an appropriate grouting program design and compaction specifications and contract documents (designer)
- Planning the grouting equipment needs and procedural approach (contractor)
- Monitoring and evaluation of the grouting program (designer and contractor)"

The pregrouting subsurface exploration is more detailed than is normally required and should include continuous sample and laboratory tests. These tests should included grain-size analysis, density, permeability, pH, and other soil index properties.

The subsurface exploration should identify the extent that grouting can be utilized and areas or site conditions where grouting cannot be utilized. Subsurface stratigraphy can be well defined by continuous sampling. Small, fine-grained lenses should be noted, since these layers can retard the progression of some types of grouting. Considerably more descriptive detail is required on the boring log to be used by a grouting specialist than is typically shown on a standard boring log. Past uses of the site should be identified, such as the presence of abandoned wells, cisterns, cesspits, etc. These items can absorb the grout and either increase the grout take or cause no ground improvement. In addition, the presence of utilities should be noted, since the bedding materials of some utilities can cause a loss of grout as well. The grouting contractor should record every anomaly encountered in the drilling and grouting operations. These anomalies should be explained and evaluated prior to continuing drilling and grouting operations. Finally, the groundwater should be well understood. Samples of the groundwater should be tested for compatibility with the grouts to be used. Different levels of pH will determine which types of grout can be used at a site. In addition, grout specimens should be prepared in the laboratory using samples of groundwater to determine if there will be any interaction between the grout and the groundwater. Further, additional samples should also be prepared using water from the actual source. The direction and rate of groundwater flow should also be established during the subsurface investigation.

19.8.3.1 Permeation Grouting

Permeation grouting uses a variety of grout materials, particulate, colloidal and solution, to permeate the soils. The choice of which grout material to use is based on the grain-size distribution of the soil to be grouted (see Figure 19-29). Permeation grouting is an option in appropriate soils for the following applications:

- Waterproofing, typically for remedial purposes
- Settlement control
- Liquefaction retrofit mitigation by increasing density and displacing pore water

For permeation grouting to be successful, the soils must be "groutable". Groutability should be based on the permeability of the soil. A first estimate of permeability, and thus groutability, is based on the fines content (i.e., the percentage of material passing the #200 sieve). Table 19-

24 and Figure 19-30 provide the approximate percentage of material passing the #200 sieve and the groutability of a soil.

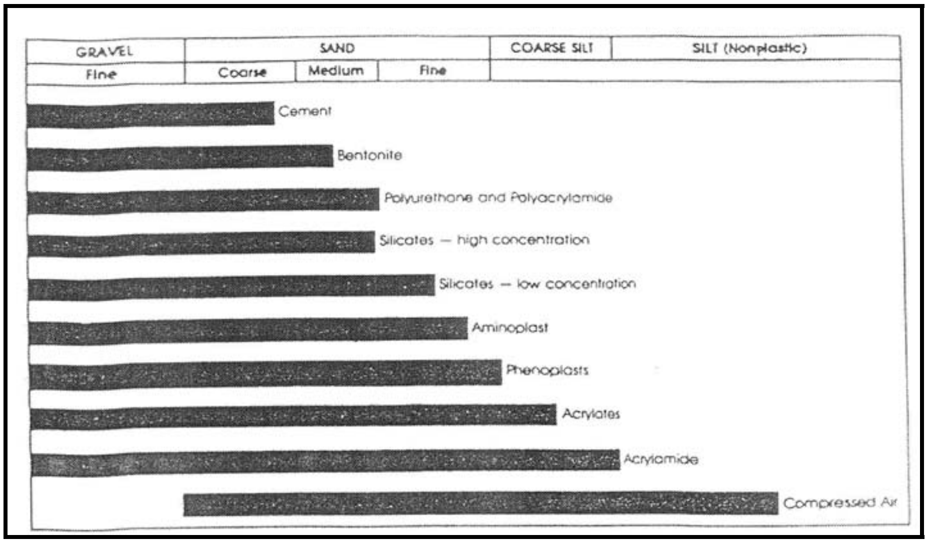


Figure 19-29, Penetrability of Various Grouts versus Soil Type (Ground Improvement Methods – August 2006)

Table 19-24, Groutability Guidelines

Percent Passing #200 Sieve	Description
<12	Readily groutable
12 – 15	Moderately groutable
15 - 20	Marginally groutable
> 20	Non-groutable

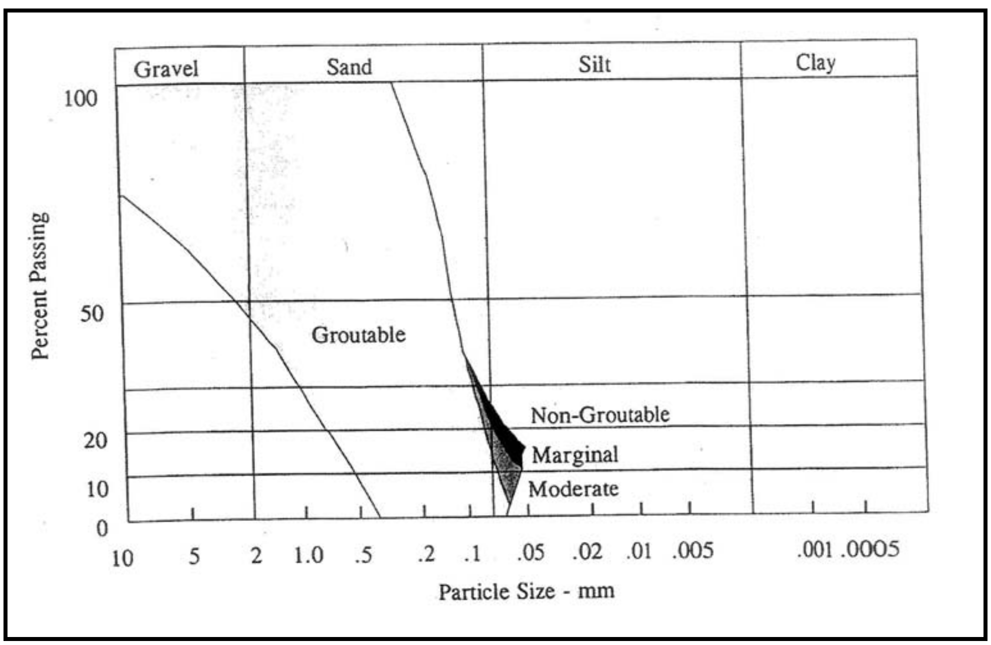


Figure 19-30, Grain-Size Distribution for Permeation Grouting (Ground Improvement Methods – August 2006)

These guidelines provide an indication of permeability; however, the actual permeability of a soil should be determined, either in the laboratory or in field pumping tests or injection tests. It should be noted that environmental permitting will be required for both pumping and injection testing. The following equations provide further guidance for the potential for permeation grouting using particulate grouts.

$$\frac{D_{15soil}}{D_{85grout}} = \Psi \qquad \text{Equation 19-23}$$

$$\frac{D_{10soil}}{D_{95grout}} = \Theta \qquad \text{Equation 19-24}$$

Where,

D_{15soil} = Diameter of the fifteen percent passing for soil
$D_{85grout}$ = Diameter of the eighty-five percent passing for the grout material
D_{10soil} = Diameter of the ten percent passing for soil
$D_{95grout}$ = Diameter of the ninety-five percent passing for the grout material

Table 19-25, Guide to Permeation Grout Potential

Groutability	Ψ	Θ
Impossible	< 11	< 6
Possible	11 – 24	6 – 11
Easy	> 24	>11

After a preliminarily determination that permeation grouting is feasible, an expert in the design of permeation grouting should be consulted to complete the final design.

19.8.3.2 Compaction Grouting

According to Ground Improvement Methods, "Compaction grouting features the use of low slump (usually 1 inch or less), low mobility grouts of high internal friction. In weak or loose soils, the grout typically forms a coherent 'bulb' at the tip of the injection pipe, thus compacting and/or densifying the surrounding soil. … If settlement has already occurred, careful compaction grouting may be used to lift and level any surface structures that have been impacted. Compaction grouts can be designed as an economic and controllable medium for helping to fill large voids, even in the presence of flowing water."

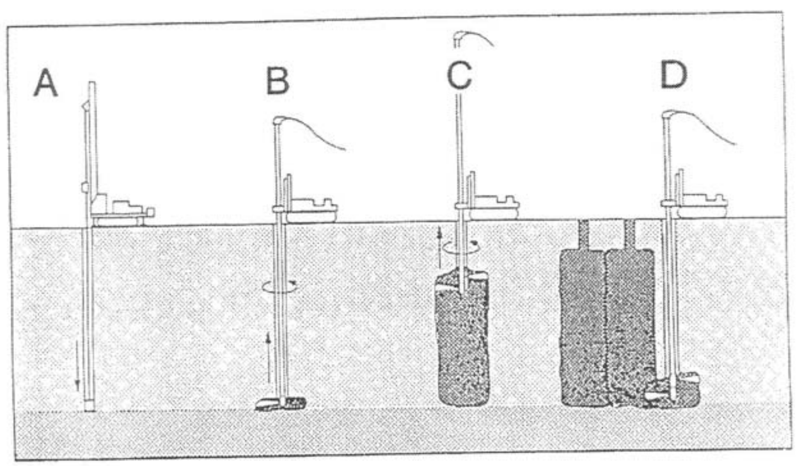
Compaction grouting has a wide variety of applications, but is primarily used in South Carolina for soil densification (for both static and seismic enhancements) and for raising surficial structures. In soil densification applications, the soils should be free-draining, such as gravels, relatively clean sands and some coarser silts (see Figure 19-30). In fine-grained soils, pore pressures may not be able to dissipate and improvement may not be achievable. In these soils, compaction grouting may displace the soil, but not cause settlement or consolidation. The grout mix design is a critical part of compaction grouting; the grout must have a high internal friction and a low slump to ensure the "bulb" forms. There are no mathematical models for use in compaction grouting (i.e., establishing the spacing, rate of injection, limiting volumes, etc.). Therefore, either an engineer or contractor that specializes in compaction grouting should be retained to assist in the final design of compaction grouting. Typically compaction grout pipes are spaced at 6-1/2 to 16-1/2 feet intervals. The amount of grout required for soil densification ranges from three to twelve percent of the soil volume being treated. Normally, compaction grouts use particulate grouts such as Portland Cement Types I or II. The slump of the compaction grout should be around 1 inch.

19.8.3.3 Jet Grouting

Jet grouting is a grouting process that uses high pressure, high velocity erosive jets of water and/or grout to remove some of the soil and replacing the removed soil with cement based grout. The combination soil and grout is called “soilcrete”. Jet grouting can be used in soils ranging from clays to gravels with varying degrees of effectiveness. Jet grouting can be used for a variety of applications:

- Water Control
- Settlement Control
- Underpinning
- Scour Protection
- Excavation Support
- Liquefaction Mitigation
- Treatment of Karst

Jet grouting permits the shape, size and properties of treated soil, usually a circular column, to be engineered in advance. Figure 19-31 provides a schematic of the jet grouting procedure.



A Drilling
B Erosion and mix-in-place operation
C Development of column-like element
D Completed elements forming a wall-like structure of interlocking elements

Figure 19-31, Jet Grouting Process Schematic
(Ground Improvement Methods – August 2006)

Jet grouting can be accomplished using three different types of jetting procedures as discussed below and depicted in Figure 19-32.

- Single Fluid System – The fluid is the grout and uses a high-pressure (7,200 psi) jet to simultaneously erode the in-situ soil and inject the grout. This system only partially replaces the soil.
- Double Fluid System – A high-pressure grout jet is contained within a compressed air cone. This system produces a larger column diameter, provides a higher degree of soil replacement, although a lower strength "soilcrete" is created.
- Triple Fluid System – An upper jet of high-pressure (4,400 to 7,200 psi) water contained inside a cone of compressed air is used for excavation, with a lower jet injects grout, at a lower pressure, to replace the slurried soil.

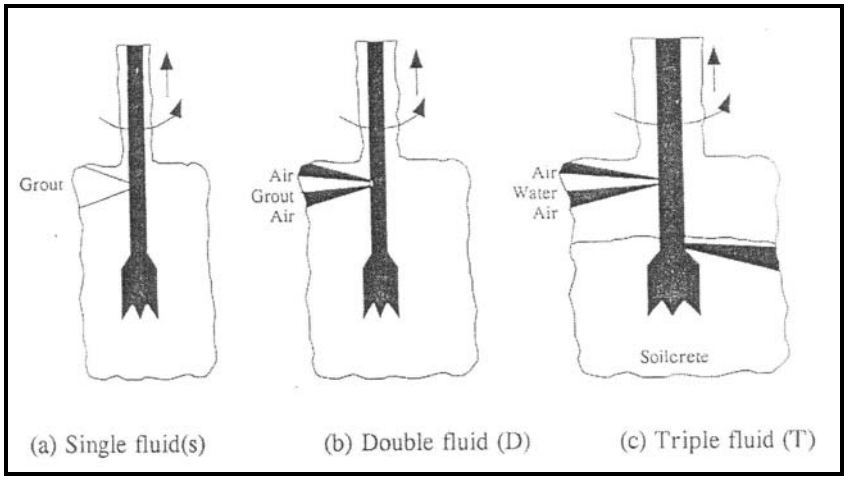


Figure 19-32, Jet Grouting Systems
(Ground Improvement Methods – August 2006)

19.8.3.4 Soil Fracture Grouting

Soil fracture grouting is the process of injecting grouts in a highly controlled manner that does not permit permeation of the grout in the soil matrix nor compaction of the soil matrix. Instead the soil matrix is ruptured and the grout forms a reinforcing "skeleton" within the matrix. Soil fracture grouting can be used to raise settled structures, control settlement, and soil reinforcement. Sophisticated measuring equipment is required when conducting this type of grouting operation. Similar to compaction grouting, designs using soil fracture grouting should be performed by an engineer or contractor specializing in this method.

19.9 COLUMN SUPPORTED EMBANKMENT

Constructing embankments over soft, compressible soils creates numerous problems (i.e., excessive settlements, embankment instability, and long times for settlements to occur). These problems have led to the development of the ground improvement methods discussed previously in this Chapter; however, in certain cases, time constraints are critical to the success of the project. Therefore, an alternate ground improvement method has been used: Column Supported Embankment (CSE) (see Figure 19-33). CSEs consist of two primary components; first, a column system to transfer loads to a more suitable bearing stratum and second, a load transfer platform (LTP). The LTP can consist of either structural concrete or a geosynthetic reinforced soil layer.

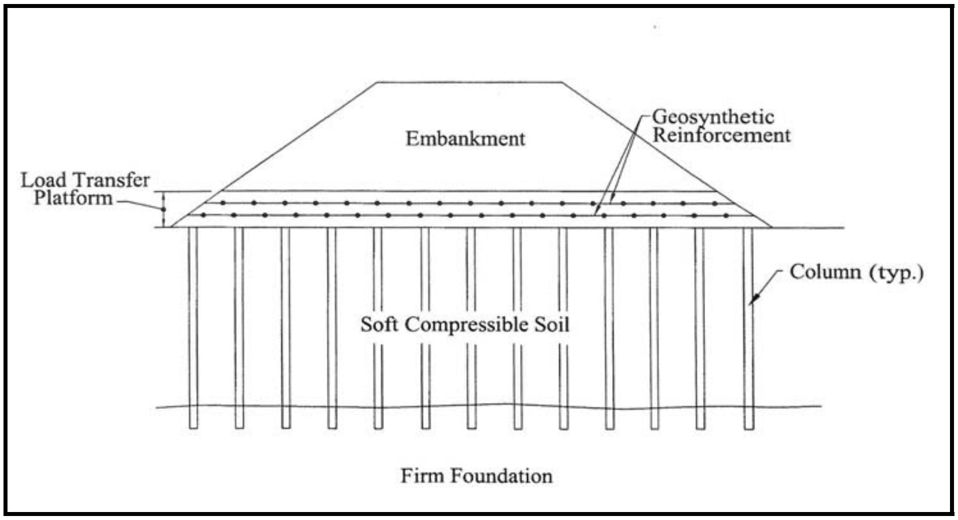


Figure 19-33, CSE with Geosynthetic LTP
(Ground Improvement Methods – August 2006)

The columns consist of typical deep foundation elements such as driven piling (prestressed concrete or steel H- or pipe piles or timber); however, the use of driven concrete or steel piling may not be economical since the capacity developed by these pile types would exceed the demand placed on the piles. These piling types should only be used if the LTP is composed of structural concrete. If the LTP is a geosynthetic reinforced soil layer, concrete and steel piling should not be used. Other types of columns can consist of timber piling, stone columns, geotextile encased columns, vibro-concrete columns, Geopiers, soil mixed columns, or continuous flight auger piles.

The LTP transfers the embankment load to the columns. The LTP may consist of either a rigid structural element or a geosynthetic reinforced soil layer. The rigid LTP is typically economically cost prohibitive and will therefore, not be discussed in this Chapter. The use of a LTP allows for the columns to be more widely spaced; however, the use of an LTP is not required if the columns are closely spaced, see Figure 19-34.

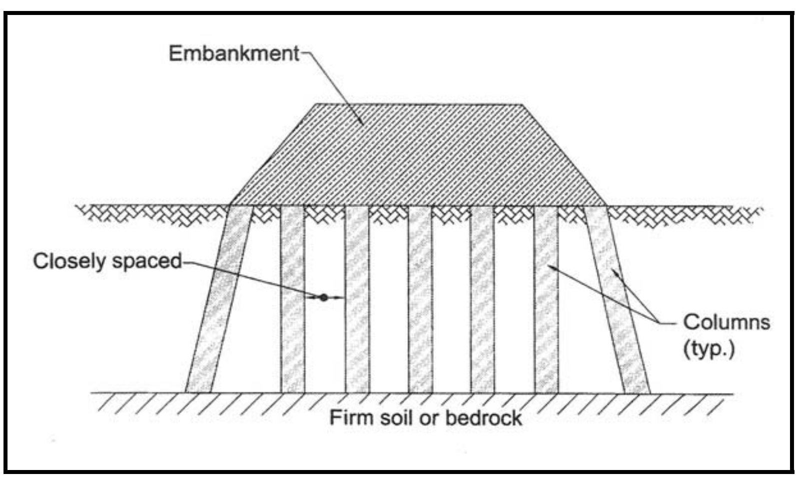


Figure 19-34, CSE without LTP
(Ground Improvement Methods – August 2006)

19.9.1 Analysis and Preliminary Design

As indicated previously, CSEs have traditionally been used to support embankments over soft soils when time constraints are such that consolidation of the soft soils is not practical. CSEs have the advantage of being constructed in a single stage. There is no waiting period for the dissipation of pore water pressures. CSEs are more economical than removing and replacing the soil, especially when the groundwater is close to the ground surface. Where infrastructure precludes high-vibration techniques, the type of column used for the CSE system may be selected to minimize or eliminate the potential for vibrations. Total and differential settlement of the embankment may be drastically reduced when using CSEs over other conventional approaches. Another benefit of using CSEs is that a variety of columns are available for support of the embankment depending on the stiffness of the subsurface soils. CSEs have the major disadvantage of having high initial costs; however, the savings in time can offset these costs. An additional disadvantage of CSEs is there is no single accepted design method. There are multiple methods and all provide different answers.

Typically, CSEs have been limited to embankment heights of approximately 35 feet. The thickness of the soft soil is not a critical component in the determination of the feasibility of using CSEs because there are a variety of columns that can be used for support. The determination of the feasibility of using CSEs should consider the following factors:

- The preliminary spacing of the columns should be limited so that the area replacement ratio is between ten and twenty percent.
- The clear span between columns should be less than the embankment height and should not exceed approximately 10 feet. Wider clear spans may lead to unacceptable differential settlement between columns.
- The fill required to create the LTP shall be select structural fill with an effective friction angle greater than or equal to 35°.
- The columns shall be designed to carry the entire load of the embankment.
- The CSE reduces post construction settlements of the embankment surface to typically less than 2 to 4 inches.

The selection of the column should also consider the potential environmental impact of the installation of the column.

19.9.2 Design

The design of CSEs is a complicated soil-structure interaction problem that requires the engineer to have a good understanding of the Strength and Service limit states of the structure. All of the design methods currently in use are empirical and primarily focus on the design of the LTP. These empirical methods include:

- The British Standard (BC8006)
- The Swedish Standard
- The German Method
- The Collin Method

The Strength limit state failure modes include the following (see Figure 19-35):

a. Failure of the columns to carry the full embankment load
b. The lateral extent of the columns must be sufficient to prevent slope instability
c. The load transfer platform must be designed to transfer the vertical load to the columns
d. Lateral sliding of the embankment on top of the columns
e. The global (overall) stability must be checked

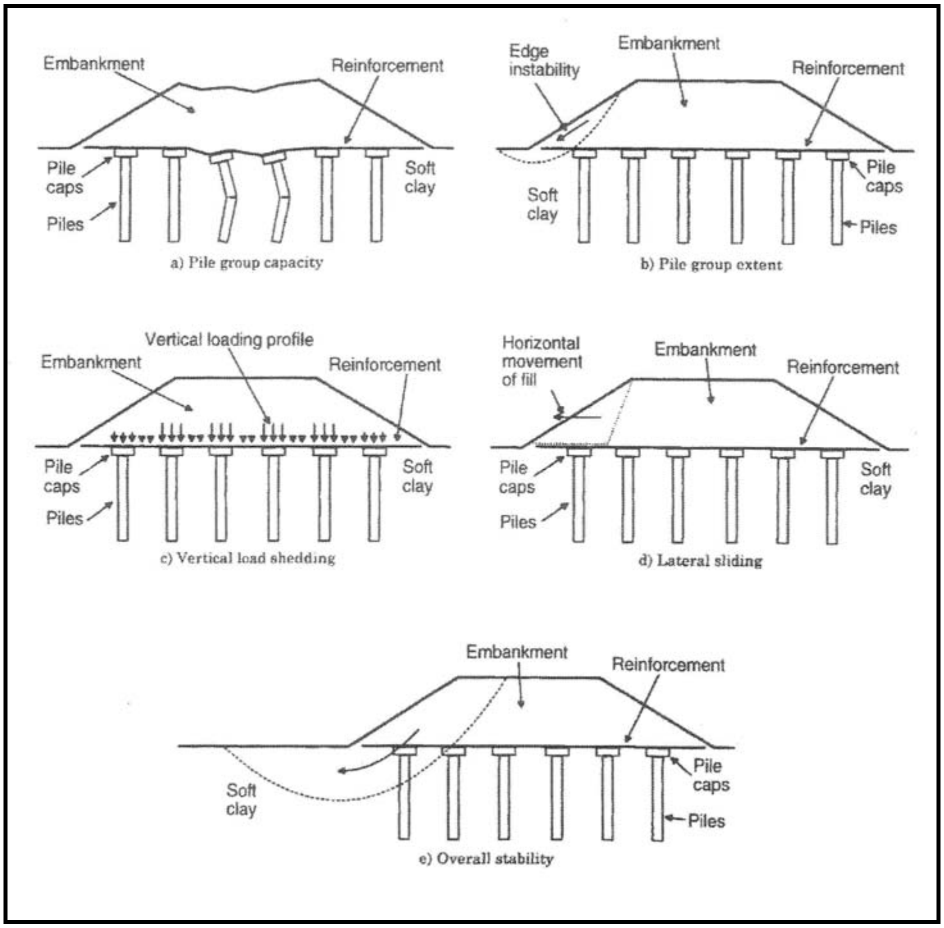


Figure 19-35, Strength Limit State Failure Modes
(Ground Improvement Methods – August 2006)

The Service limit state of the CSE must also be checked. The strain in the geosynthetic reinforcement used to create the LTP should be kept below some maximum threshold to preclude unacceptable deformation reflection (see Figure 19-36, Detail a) at the top of the embankment. In addition, the settlement of the columns should also be analyzed to ascertain whether the CSE will develop unacceptable settlements (see Figure 19-36, Detail b).

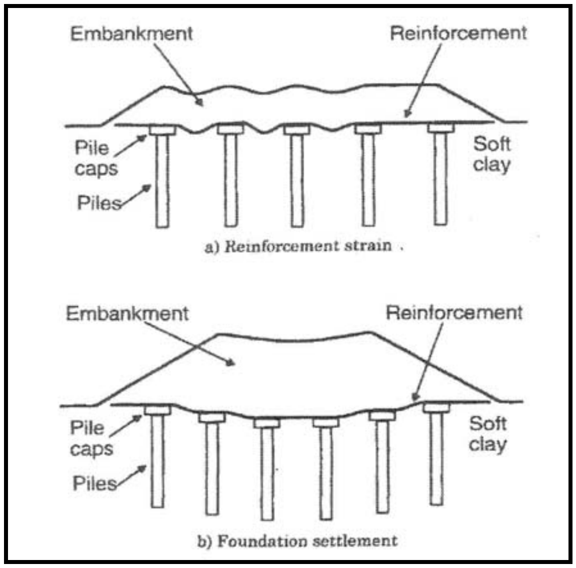


Figure 19-36, Service Limit State
(Ground Improvement Methods – August 2006)

The general design procedure for CSEs is provided below:

1. Estimate preliminary column spacing (see previous Section)
2. Determine required column load
3. Select preliminary column type based on required column load and site geotechnical requirements
4. Determine capacity of column to satisfy Strength and Service limit state design requirements
5. Determine extent of columns required across embankment width
6. Select LTP design approach (catenary or beam)
7. Determine reinforcement requirements based on estimated column spacing.
8. Revise column spacing as required.
9. Determine reinforcement requirements for lateral spreading.
10. Determine overall reinforcement requirements based on LTP and lateral spreading.
11. Check global stability.
12. Prepare construction drawings and specifications.

19.9.2.1 Column Design

The selection of the type of column should be based on the constructability, load capacity, and cost of the various column types. The load carrying capacity of each column is based on the tributary area of each column (see Figure 19-37). In CSE design, it is assumed that the weight of the embankment and any surcharge loads are carried by the columns and that the surrounding soil carries minimal, if any, load. The tributary area for a single column is geometrically a hexagon; however, for simplification a circle having the same tributary area is used. Figure 19-37 provides the effective diameter (D_e) for both triangular and square spacings. The typical center-to-center column spacing is 5 to 10 feet. The required design vertical load (Q_r) in the column is determined by the following equation:

$$\boldsymbol{Q_r} = \pi \left(\frac{\boldsymbol{D_e}}{\boldsymbol{2}} \right)^{\boldsymbol{2}} (\gamma \boldsymbol{H} + \boldsymbol{q})$$ Equation 19-25

Where,

D_e = Effective tributary area of column
H = Height of embankment
γ = Unit weight of embankment soil
q = Live and dead load surcharge (determined similar to long-term stability analysis)

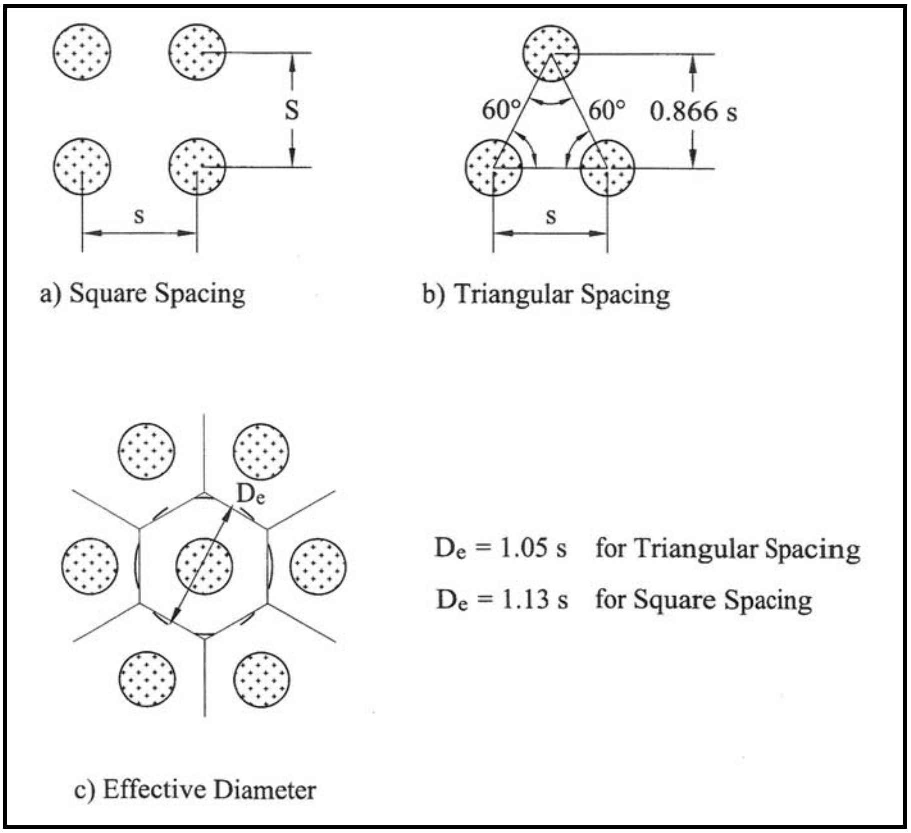


Figure 19-37, CSE Column Layout
(Ground Improvement Methods – August 2006)

In addition to determining the load to be carried by the columns, the lateral extent of the columns will also need to be determined. The columns should extend a sufficient distance beyond the crest of the embankment to ensure that any instability or differential settlement that occurs beyond the limits of the columns will not affect the crest of the embankment. The extent of the columns should be determined using a slope stability program (see Chapter 17). For preliminary designs and feasibility studies, the following equations from the British Standard (BS8006) may be used.

$$\boldsymbol{L_p} = \boldsymbol{H}(\boldsymbol{n} - \tan \theta_{\boldsymbol{p}})$$ Equation 19-26

$$\theta_p = \left(45 - \frac{\varphi'_{emb}}{2}\right)$$ Equation 19-27

Where,

L_p = Horizontal distance from the toe of the embankment to the edge of first column

n = Side slope of embankment (see Figure 19-38)

θ_p = Angle from vertical between the outer-most column and the crest of the embankment (see Figure 19-38)

φ'_{emb} = Effective friction angle of embankment fill

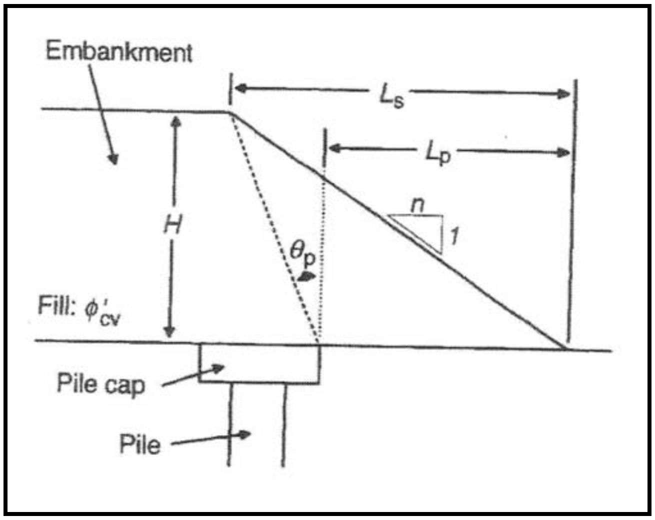


Figure 19-38, CSE Edge Stability
(Ground Improvement Methods – August 2006)

The potential for lateral spreading of the embankment must be analyzed (Figure 19-39). The geosynthetic reinforcement must be designed to prevent lateral spreading of the embankment. This is a critical aspect of the design, because many columns used to support CSEs are not capable of developing adequate lateral resistance to prevent the spreading of the embankment. The geosynthetic reinforcement must be designed to resist the horizontal force caused by the lateral spreading of the embankment. The required tensile force to prevent lateral spreading (T_{ls}) is determined using the following equations.

$$T_{ls} = \frac{K_a(\gamma H + q)H}{2}$$ Equation 19-28

$$K_a = \tan^2\left(45 - \frac{\varphi'_{emb}}{2}\right)$$ Equation 19-29

The minimum length of reinforcement (L_e) required to prevent the sliding of the embankment across the reinforcement is determined using the following equation.

$$L_e = \frac{T_{ls}}{[0.5\gamma H(c_{iemb}\tan\varphi'_{emb})]}$$ Equation 19-30

Where,

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

φ'_{emb} = Friction angle of embankment fill material (φ'_{cv} in figure 19-39

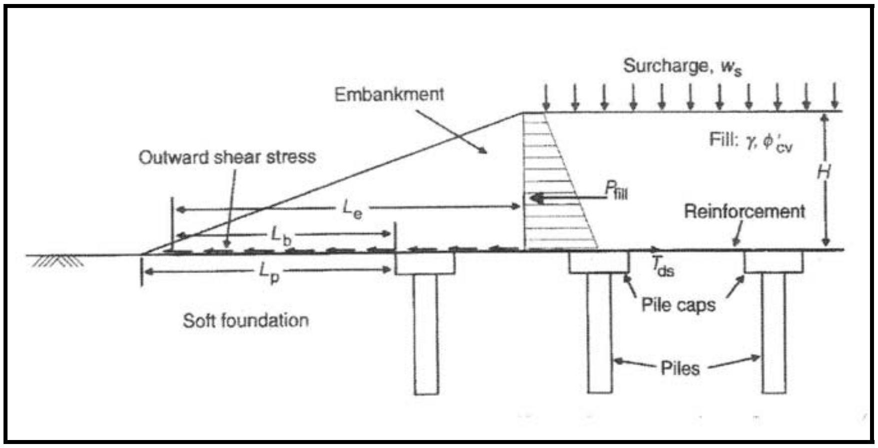


Figure 19-39, CSE Lateral Spreading
(Ground Improvement Methods – August 2006)

19.9.2.2 Load Transfer Platform

The load transfer platform (LTP) has two design approaches (catenary or beam) (see Figure 19-40). The three methods for design of the LTP using the catenary design approach are the British Standard (BS 8006), the Swedish Method and the German Method. The catenary approach makes the following assumptions:

- Soil arch forms in the embankment
- Reinforcement is deformed during loading
- One layer of reinforcement is used

The beam design approach has one design method, the Collin. The beam approach makes the following assumptions

- A minimum of three layers of reinforcement are used to create the platform
- Spacing between the layers of reinforcement is 8 to 18 inches
- The platform thickness is greater than or equal to one-half of the clear span between columns (edge-to-edge)
- Soil arch is fully developed with the depth of the platform

The catenary approach normally requires higher strength reinforcement for the same design conditions, as opposed to the beam approach. The beam approach will typically allow for larger column-to-column spacing than the catenary approach for standard geosynthetics (i.e., materials available off the shelf).

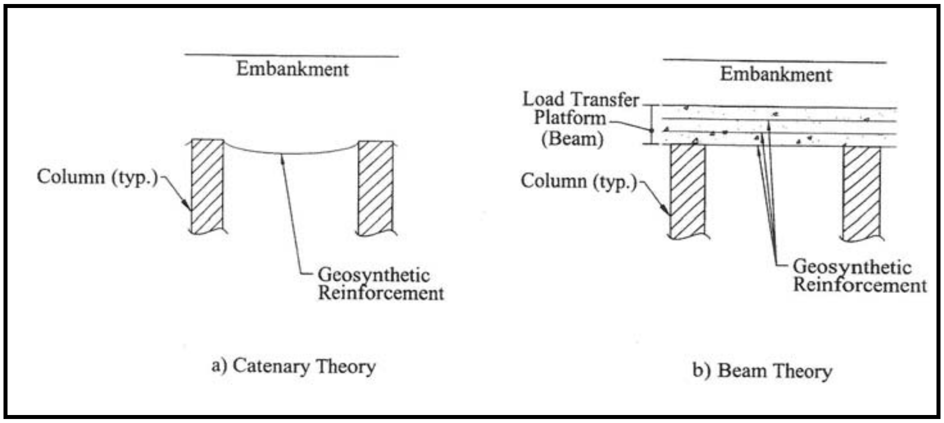


Figure 19-40, Load Transfer Mechanisms
(Ground Improvement Methods – August 2006)

Both design approaches rely on the soil forming an arch. Soil arching, according to Ground Improvement Methods, is "the ability of material to transfer loads from one location to another in response to a relative displacement between locations." Figure 19-41 provides schematics of soil arching.

According to Ground Improvement Methods, "The stress at point "a" in Figure *19-41, Detail* a, is equal to the overburden stress γH, where γ is the unit weight of the soil, and H is the height of the soils mass. At the moment when the soil loses support, a temporary true arch is formed. The soil at point "a" is in tension, and the weight of the soil prism starts to be transferred to the adjacent unyielding soil (Figure *19-41, Detail* b). Deformation within the temporary true soil arch occurs. As the soil settles into an inverted arch (Figure *19-41, Detail* c), an equilibrium state is achieved, the adjacent unyielding soil mobilizes its shear strength, and the load transfer is complete. At some height (H_e) above point "a," the transfer of stress is complete. The settlements in the soil mass above this point are uniform. The degree of soil arching is defined as the soil arch ratio (ρ), which is the ratio of the average vertical stress on the yielding portion (i.e., soft soil between columns) to the average vertical stress due to the embankment fill and surcharge load."

$$\rho = \frac{\sigma_s}{(\gamma_{emb} H + q)} \qquad \text{Equation 19-31}$$

Where,

H = Height of embankment

γ_{emb} = Unit weight of embankment material

q = Live and dead load surcharge (determined similar to long-term stability analysis)

σ_s = Average vertical stress applied between columns

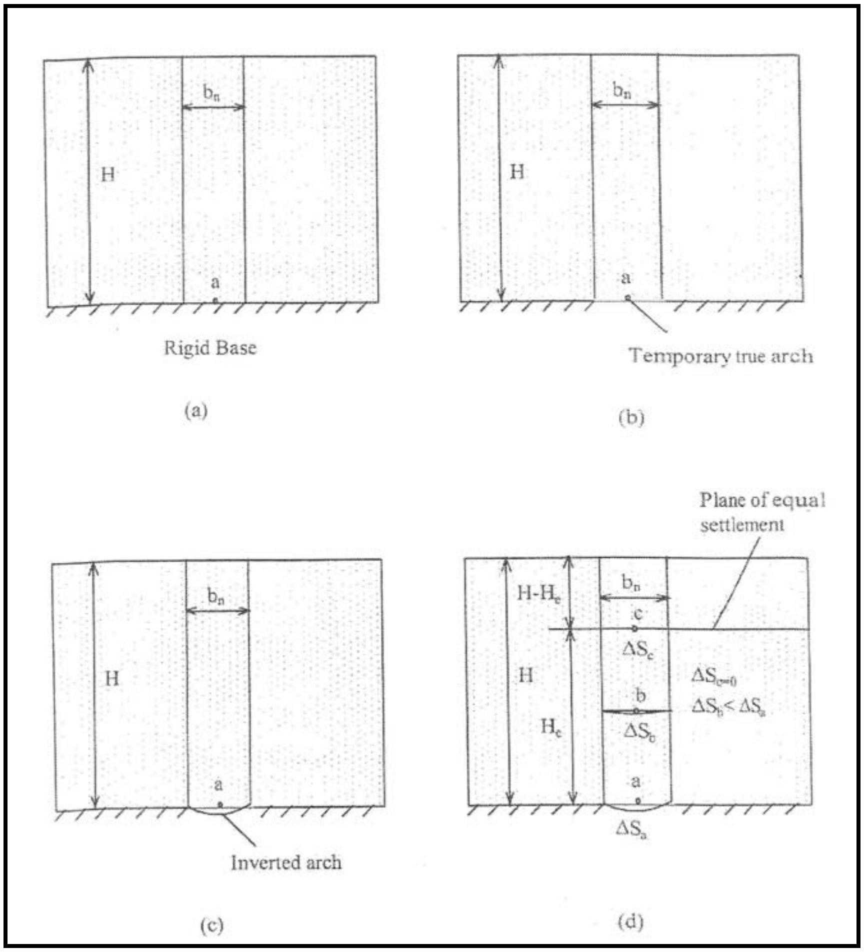


Figure 19-41, Soil Arching
(Ground Improvement Methods – August 2006)

In addition to soil arching, the load transfer platform design relies on the geosynthetic developing resistance to tension (i.e., tension membrane theory). The vertical load from the soil within the arch and any surcharge load, if the thickness of the embankment is not great enough to develop the full arch, is carried by the reinforcement.

The variables and symbols used by all of the design methods have been standardized. Figure 19-42 depicts the common symbols that will be used in presenting each of these methods; further each variable and/or symbol is defined also.

Where,

d = Diameter of column
H = Height of embankment
P_c' = Vertical stress on column
q = Surcharge load
s = Center-to-center column spacing
T_{RP} = Tension in the extensible reinforcement
W_T = Vertical load carried by the reinforcement
γ_{emb} = Unit weight of embankment material
φ'_{emb} = Effective friction angle of the embankment fill material

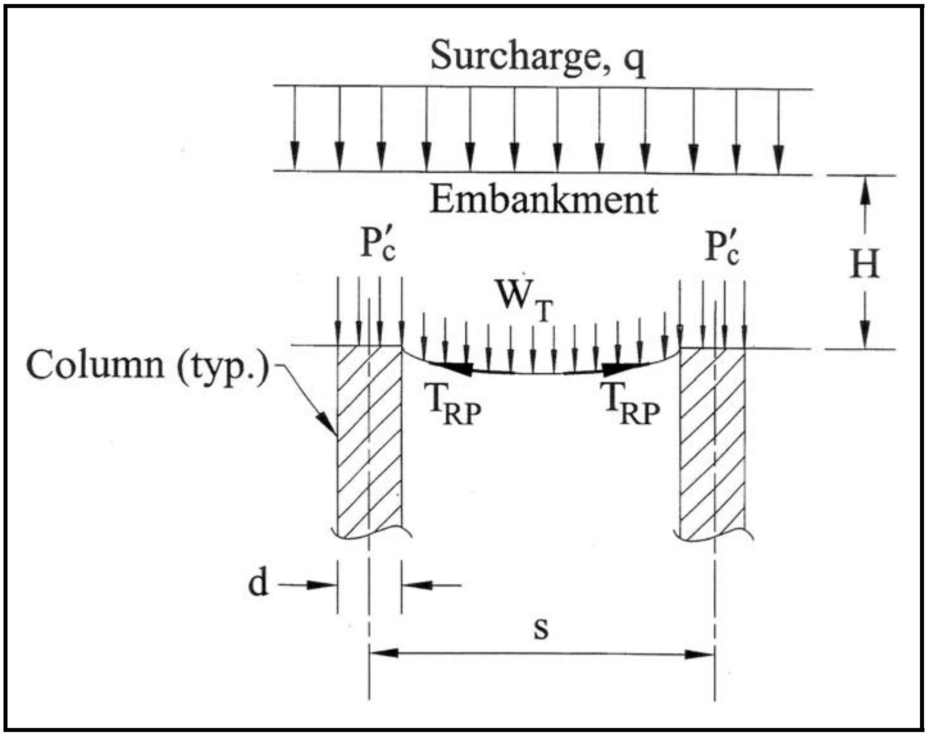


Figure 19-42, CSE Definition of Terms
(Ground Improvement Methods – August 2006)

19.9.3 Catenary Design Approach

The catenary design approach depends on the arching effect of the soil and the ability of the geosynthetic reinforcement to resist in tension the load applied by the embankment and surcharge. There are three design methods that use the catenary design approach:

- British Standard (BS 8006)
- Swedish Method
- German Method

19.9.3.1 British Standard (BS 8006)

To ensure that differential settlement does not occur at the surface of the embankment, the British Standard recommends that the embankment height be a minimum of 1.4 times the clear span (edge-to-edge) between columns. At this height, the soil arches and the columns carry more load than the surrounding soil. The ratio of vertical stress on the columns to the average vertical stress at the base of the embankment is determined from the following equations.

$$\frac{P'_c}{\sigma'_c} = \left[\frac{C_c d}{H}\right]^2$$ Equation 19-32

$$\sigma'_c = \left(f_{fs}\gamma_{emb}H + f_q q\right)$$ Equation 19-33

Where,

σ'_c = Average vertical stress at the base of the embankments
f_{fs} = Partial soil unit mass load factor (1.3)
f_q = Partial surcharge load factor (1.3)
C_c = Arching coefficient
= [(1.95H/d)-0.18] for end bearing columns (unyielding)
= [(1.50H/d)-0-07] for frictional columns (normal)

The vertical load (W_T) carried by the reinforcement is determined from the following equations:

Table 19-26, Vertical Load Determination Equations

H	W_T	
< 0.7 (s-d)	N/A	
0.7 (s-d) ≤ to ≤ 1.4 (s-d)	$\left[\frac{s(f_{fs}\gamma H + f_q q)}{(s^2 - d^2)}\right]\left[s^2 - d^2\left(\frac{P'_c}{\sigma'_c}\right)\right]$	Equation 19-34
> 1.4 (s-d)	$\left[\frac{1.4 s f_{fs}\gamma(s-d)}{(s^2 - d^2)}\right]\left[s^2 - d^2\left(\frac{P'_c}{\sigma'_c}\right)\right]$	Equation 19-35

The tension in extensible reinforcement (T_{RP}) per linear foot of reinforcement resulting from the distributed load is determined using the following equation.

$$T_{RP} = 0.5 W_T \left[\frac{(s-d)}{d}\right]\sqrt{\left(1 + \frac{1}{6\varepsilon}\right)} \qquad \text{Equation 19-36}$$

Where,

ε = Strain in the reinforcement

Some initial strain is required to generate a tensile force in the reinforcement; however, the practical upper limit on strain is six percent. Using this limit ensures that the embankment load is transferred to the columns. The long-term strain in the reinforcement (caused by creep) should be limited to ensure that the long-term localized deformations (dimples) do not occur at the ground surface. A minimum creep strain of two percent over the design life (100 years) of the reinforcement is allowed.

19.9.3.2 Swedish Method

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

- Arch formation occurs
- Reinforcement is deformed during loading
- One layer of reinforcement is used
- Reinforcement is located within 4 inches of the column
- The embankment height is greater than or equal to the clear distance between the columns (edge-to-edge)

- The ratio of column area to the influence area per column is greater than or equal to ten percent
- The embankment fill effective friction angle is 35°
- Initial strain in the reinforcement is limited to six percent
- Long-term (creep) strain is limited to two percent
- Total strain is less than seventy percent strain at failure

The model used in the Swedish Method to determine the vertical load is provided in Figure 19-43. The cross sectional area of the soil under the arch, which is the load carried by the reinforcement, is approximated by the soil wedge shown in Figure 19-43. This applies even when the embankment height is lower than the top of the soil wedge. The height of the soil wedge is determined from the following equation:

$$\boldsymbol{h} = \frac{(\boldsymbol{s} - \boldsymbol{d})}{\mathbf{2\,tan15^{\circ}}} \qquad \text{Equation 19-37}$$

Where,

h = Height of soil wedge or arch
s = Center-to-center column spacing
d = Diameter of column

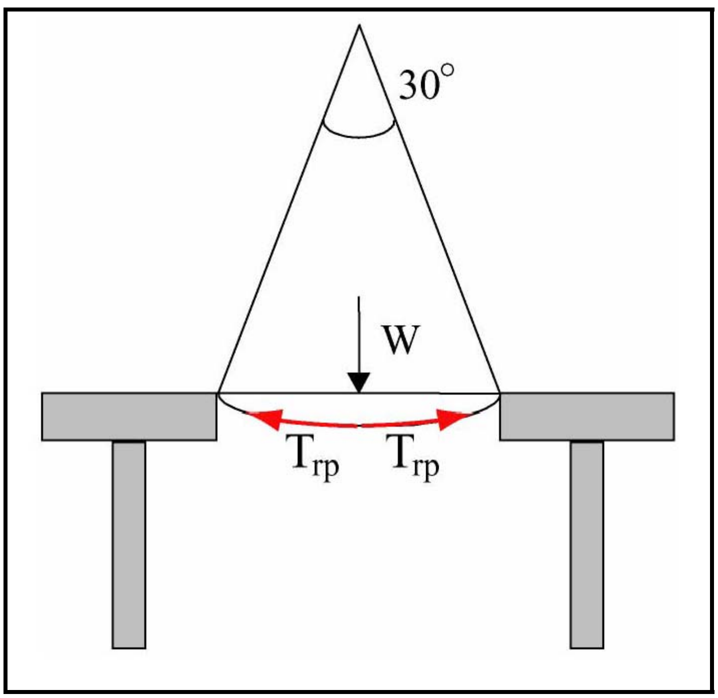


Figure 19-43, Swedish Method Model
(Nordic Guidelines for Reinforced Soils and Fills – February 2004)

The two-dimensional weight (W_T) of the soil wedge per unit length in depth is determined from the following equation.

$$\boldsymbol{W_T} = \frac{(\boldsymbol{s} - \boldsymbol{d})^2 \gamma_{emb}}{\mathbf{4\,tan15^{\circ}}} \qquad \text{Equation 19-38}$$

The force in the reinforcement, per unit length in depth, due to the two-dimensional weight (W_T) in three-dimensions is determined using the following equation.

$$T_{RP} = 0.5W_T\left[1+\left(\frac{s}{d}\right)\right]\sqrt{1+\frac{1}{6\varepsilon}} \qquad \text{Equation 19-39}$$

19.9.3.3 German Method

The German Method considers the effect of the soft foundation soil in determining the load carrying capability of the reinforcement, unlike either the British Standard or Swedish Methods, which do not. The German Method considers both the undrained shear strength of the foundation material, as well as the shear strength of the embankment material. This method is only applicable if the height of the embankment is greater than the column spacing. Two failure criteria are considered:

1. Failure of the embankment fill at the crown of the arch
2. Failure at the bearing point of the arch

The ratio (E) of the vertical load on the columns to the average load at subgrade is a function of which failure mode controls the design. Failure at the crown of the arch occurs for relatively shallow embankments with wide column spacing. E is determined from the following equations:

$$E = 1-\left[1-\left(\frac{d}{s}\right)^2\right](A - AB + C) \qquad \text{Equation 19-40}$$

$$A = \left[1-\left(\frac{d}{s}\right)\right]^{2(K_p-1)} \qquad \text{Equation 19-41}$$

$$B = \left[\frac{s}{1.41H}\right]\left[\frac{(2K_p-2)}{(2K_p-3)}\right] \qquad \text{Equation 19-42}$$

$$C = \left[\frac{(s-d)}{1.41H}\right]\left[\frac{(2K_p-2)}{(2K_p-3)}\right] \qquad \text{Equation 19-43}$$

$$K_p = \frac{(1+\sin\varphi'_{emb})}{(1-\sin\varphi'_{emb})} = \tan^2\left(45+\frac{\varphi_{emb}'}{2}\right) \qquad \text{Equation 19-44}$$

Failure at the bottom of the arch is determined using the following equations.

$$E = \frac{\beta}{(1+\beta)} \qquad \text{Equation 19-45}$$

$$\beta = \left[\frac{2K_p}{(K_p + 1)\left(1 + \frac{d}{s}\right)}\right]\left[\left(1 - \frac{d}{s}\right)^{-K_p} - \left(1 + \frac{K_p d}{s}\right)\right] \qquad \text{Equation 19-46}$$

The minimum value of E controls the stress applied to the soil between the columns (σ_s). The stress that is applied to the soil between columns (Figure 19-44) is determined using the following equation.

$$\sigma_s = \left[\frac{(\gamma_{emb}H + q)}{(s^2 - d^2)}\right][1 - E]s^2 \qquad \text{Equation 19-47}$$

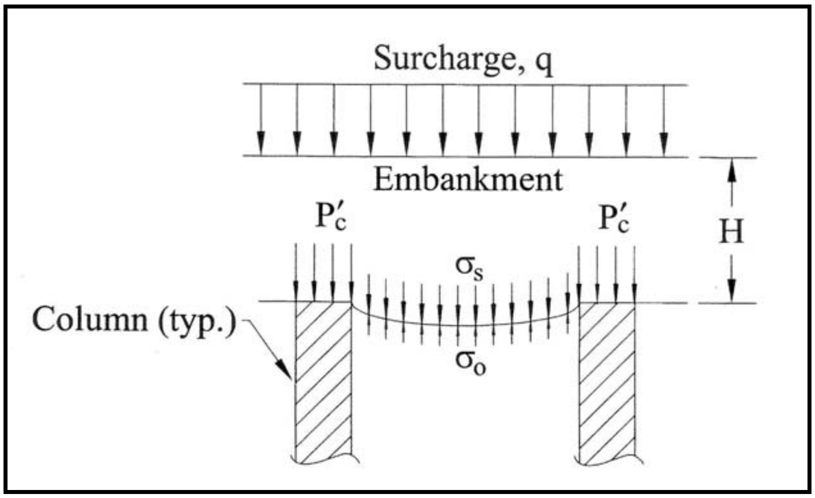


Figure 19-44, German Method Model
(Ground Improvement Methods – August 2006)

The geosynthetic reinforcement carries the stress imposed by the embankment (σ_s) minus the resistance of the soil (σ_o) located between the columns. The resistance of the soil (σ_o) between the columns is determined using the following equation.

$$\sigma_o = 0.5[(2 + \pi)c_u] \qquad \text{Equation 19-48}$$

Where,

c_u = Undrained shear strength of the soil between the columns

0.5 = Resistance Factor (φ) used to determine σ_o

The vertical load (W_T) per unit length on the geosynthetic reinforcement spanning between the columns is determined using the following equation.

$$W_T = \left[\frac{\sigma_s(s^2 - d^2)}{2(s' - d)}\right] - \left[\frac{\sigma_o(s^2 - d^2)}{2(s' - d)}\right] \qquad \text{Equation 19-49}$$

Where,

s' = s for square column pattern

s' = 1.4s for triangular column pattern

The German Method assumes that the geosynthetic reinforcement is placed less than 1-1/2 feet above the columns. The tensile force in the reinforcement per unit length of reinforcement is determined based on catenary tension and is determined using the following equation.

$$T_{RP} = W_T \left[\frac{(s'-d)}{2d} \right] \left(\sqrt{1 + \frac{1}{6\varepsilon}} \right)$$ Equation 19-50

19.9.4 Beam Design Approach

The beam design approach consists of one method, the Collin Method, and is fundamentally different from the catenary design approach. The beam design approach is based on the premise that the reinforcement creates a stiffened beam of reinforced soil to distribute the load imposed by the embankment to the columns. The stiffened beam of reinforced soil should contain a minimum of three layers of reinforcement (Figure 19-45). This beam acts as the LTP for the Collin Method.

The Collin Method is based on the following assumptions:

- The thickness (h) of the LTP is equal to or greater than one-half of the clear span between the columns (i.e., 0.5(s-d))
- A minimum of three layers of geosynthetic reinforcement is used to create the LTP
- A minimum distance of 8 inches is maintained between the layers of reinforcement
- Select fill is used to construct the LTP
- The primary function of the reinforcement is to provide lateral confinement of the select fill to facilitate soil arching within the thickness (h) of the LTP
- The secondary function of the reinforcement is to support the wedge of the soil below the arch
- All of the vertical load from the embankment above the load transfer platform is transferred to the columns below the platform
- The initial strain in the reinforcement is limited to five percent

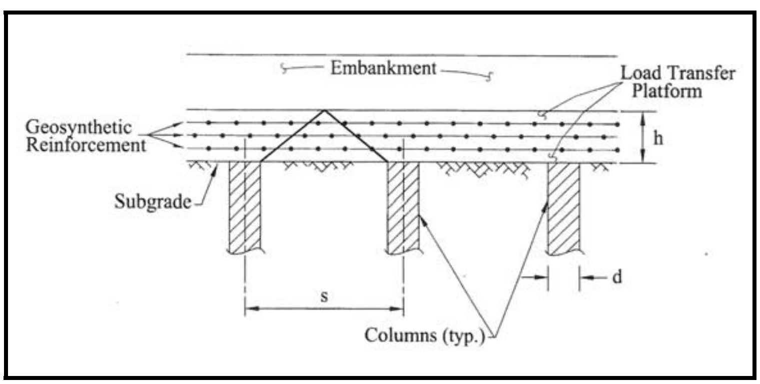


Figure 19-45, Load Transfer Platform
(Ground Improvement Methods – August 2006)

The fill load attributed to each layer of reinforcement is the material located between the layer of reinforcement and the next layer above (Figure 19-46). The uniform vertical load on any layer (n) of reinforcement (W_{Tn}) may be determined using the following equations for a triangular pattern and a square pattern, respectively.

$$W_{Tn} = \frac{\left((s-d)_n^2 + (s-d)_{n+1}^2\right)\sin 60^\circ h_n \gamma_{emb}}{(s-d)_n^2 \sin 60^\circ}$$ Equation 19-51

$$W_{Tn} = \frac{\left((s-d)_n^2 + (s-d)_{n+1}^2\right) h_n \gamma_{emb}}{(s-d)_n^2}$$ Equation 19-52

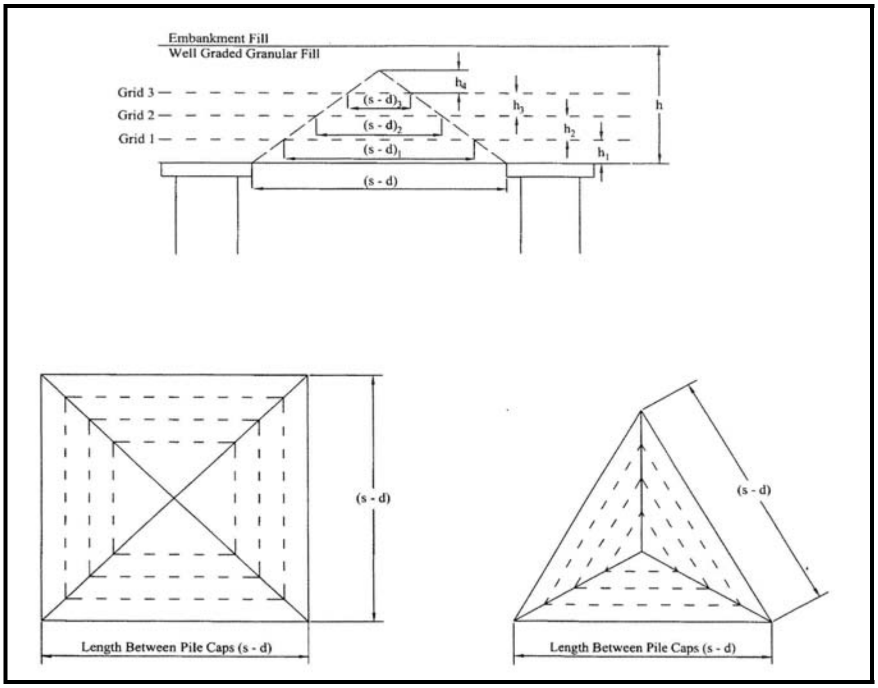


Figure 19-46, Collin Method Load Transfer Platform Design (Ground Improvement Methods – August 2006)

The tensile load on any layer of reinforcement (T_{RPn}) is determined based on tension membrane theory and is a function of the amount of strain in the reinforcement. T_{RPn} is determined using the following equation:

$$T_{RPn} = W_{Tn} \Omega D / 2$$ Equation 19-53

Where,

D = $(s-d)_n$ for square column spacing

D = $(s-d)_n$tan30 for triangular column spacing

Ω = From Table 19-27

Table 19-27, Values of Ω
(Ground Improvement Methods – August 2006)

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

19.9.5 Reinforcement Total Design Load

Regardless of the method used to design the LTP, the maximum design load (T_{max}) on the geosynthetic reinforcement is determined using the following equations:

Reinforcement along the length of the embankment (longitudinal direction of road)

$$T_{\max} = T_{RP} \qquad \text{Equation 19-54}$$

Reinforcement across the width of the embankment (transverse direction of road)

$$T_{\max} = T_{RP} + T_{ls} \qquad \text{Equation 19-55}$$

19.10 REINFORCED EMBANKMENTS ON SOFT FOUNDATIONS

Embankments constructed on soft soil foundations have a tendency to move both in the vertical as well as the horizontal directions. The vertical settlements are dealt with using ground improvement methods discussed previously in this Chapter. The horizontal movements can consist of either a general sliding of the embankment (block type failure) or from lateral squeeze (see Chapter 17). As indicated in Chapter 17, the soft soils will gain strength with time due to the settlement, however, some reinforcement of the subgrade may be required to prevent lateral movements or slope instabilities while the subgrade soils are gaining strength. There are two uses of reinforced embankments on soft soil foundations. The first is as an aide to construction and the second is reinforcement of the slope.

The use of reinforced embankments over soft foundation soils typically fall into two situations; first, construction over uniform deposits, and second, construction over localized anomalies (Figure 19-47). The most common application in transportation construction is the placement of embankments over uniform soft soil foundations. Typically, the reinforcement is placed perpendicular to the centerline of the embankment to prevent long joints parallel to the centerline and the potential for sliding of the outer most reinforcement. As the end of the embankment is approached, the turning of the reinforcement may be required.

The reinforcement normally used consists of biaxial and uniaxial geogrids; however, geotextiles may also be used. The design using geotextiles is based upon constructability, survivability, and the amount of strain required to achieve the desired strength.

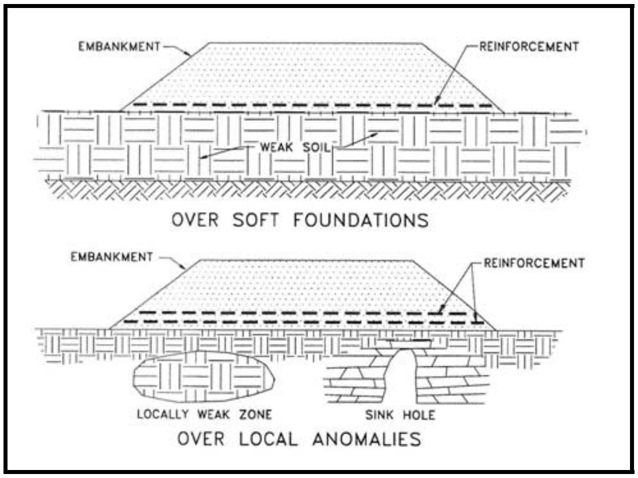


Figure 19-47, Reinforced Embankment Applications (Ground Improvement Methods – August 2006)

19.10.1 Subgrade Stabilization

The use of reinforcement beneath an embankment as subgrade stabilization is also called "bridging". Bridging is only required if the in-situ soil has an undrained shear strength ($\tau = c_u$) less than 500 pounds per square foot or 3.5 pounds per square inch. A bridge lift should be considered if the exposed subgrade soils are susceptible to deterioration (i.e. contain plastic fines) from inclement weather and exposure to vehicular traffic. Basically, the reinforcement is not considered as part of the design of the embankment, but is placed exclusively to permit construction to proceed, by stabilizing the subgrade materials to permit the placement of bridging materials. Further, the use of reinforcement and bridging materials will not prevent or mitigate settlement or slope instability; other ground improvement methods are required to mitigate settlement or slope instability. The reinforcement typically consists of either a geogrid or a geotextile. The use of the reinforcement is to limit the amount of excavation (undercutting or mucking) required. The standard construction practice using reinforcement to aide construction is presented below.

1. Muck excavation to required depth (if necessary)
2. Placement of reinforcement and/or soil separator (if necessary)
3. Placement of bridge lift
4. Placement of soil separator (if necessary)
5. Placement and compaction of backfill materials

The bridge lift may consist of either stone (No. 57 or No. 67) or granular materials (A-1-a, A-1-b, A-3, A-2-4, A-2-5, A-2-6) and is not normally compacted to the level required for the remainder of the embankment. Bridge lift materials placed in water should consist of stone or coarse granular materials (A-1-a). If mucking is performed below the water level, the bridge lift is normally placed to at least 6 inches above high water, regardless of the bridge thickness determined; a soil separator is placed on top of the bridge lift as necessary. The thickness of the bridge lift is determined using both the US Forest Service (Steward, et. al.) and the Giroud-Han methods as presented in Geosynthetic Design and Construction Guidelines. The thickest bridge lift shall be used in design. The top of the bridge lift shall not be closer than 3 feet beneath the bottom of the pavement structure, unless the bridge lift is constructed of stone.

19.10.1.1 US Forest Service (Steward, et. al.) Method

The US Forest Service (USFS) Method is a chart based solution that requires knowledge of not only the soil conditions, but the methods of fill placement and sizes of construction equipment. Since it will be practically impossible to ascertain the type and size of construction equipment to be used, the type and size of construction equipment should be indicated on the drawings as a limitation until at least 3 feet of embankment fill has been placed. This method is applicable to both geotextiles and geogrids.

The first step in using the USFS Method is determining the subgrade strength, the undrained shear strength (c_u, τ (psi)) should be determined from either CPT or DMT soundings or from field vane shear tests. Undrained shear strength, in psi, may also be estimated from field CBR values using the following equation.

$$c_u = 4.3(CBR) \qquad \text{Equation 19-56}$$

The second and third steps handle the anticipated traffic configuration. The type of construction equipment anticipated should be indicated as well as the amount traffic passes. It should be noted that the minimum number of traffic passes is 100, while the maximum is 1,000. It should be noted that the traffic estimate is based on the vehicles having a tire pressure of 80 psi. In the fourth step the depth of the tolerable rut is determined. The depth of the tolerable rut ranges from 2 to 4 inches.

The fifth step is determining the bearing capacity factor (N_c) for both the condition of without reinforcement and with reinforcement. The table below provides the bearing capacity factor based on the reinforcement condition, tolerable rut depth, and traffic.

Table 19-28, Bearing Capacity Factors for USFS Method (adopted from Geosynthetics Engineering – August 2008)

Reinforcement	Tolerable Rut (inches)	Traffic (18 kip ESALs)	Bearing Capacity Factor (N_c)
Without	< 2	> 1,000	2.8
	2 to 4	100 – 1,000	3.0
	> 4	< 100	3.3
Geotextiles	< 2	> 1,000	5.0
	2 to 4	100 – 1,000	5.5
	> 4	< 100	6.0
Geogrids	< 2	> 1,000	5.8

Step six consists of determining the amount of bridge lift material required for both the unreinforced as well as the reinforced subgrade. The material thicknesses are determined from Figures 19-48 for single wheel loads and 19-49 for dual wheel loads depending on the vehicular configuration assumed in the third step.

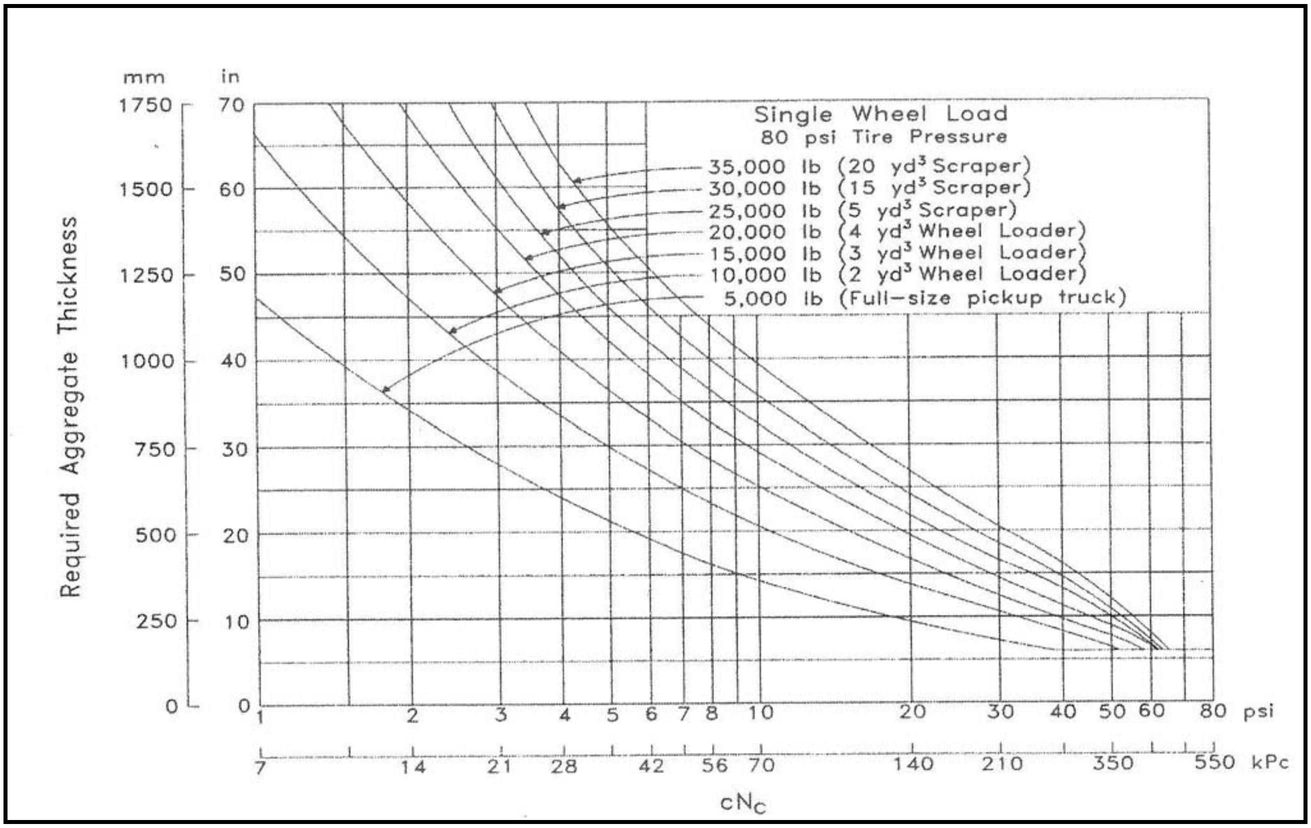


Figure 19-48, USFS Method Bridge Lift Thickness – Single Wheel Loads (adopted from Geosynthetics Engineering – August 2008)

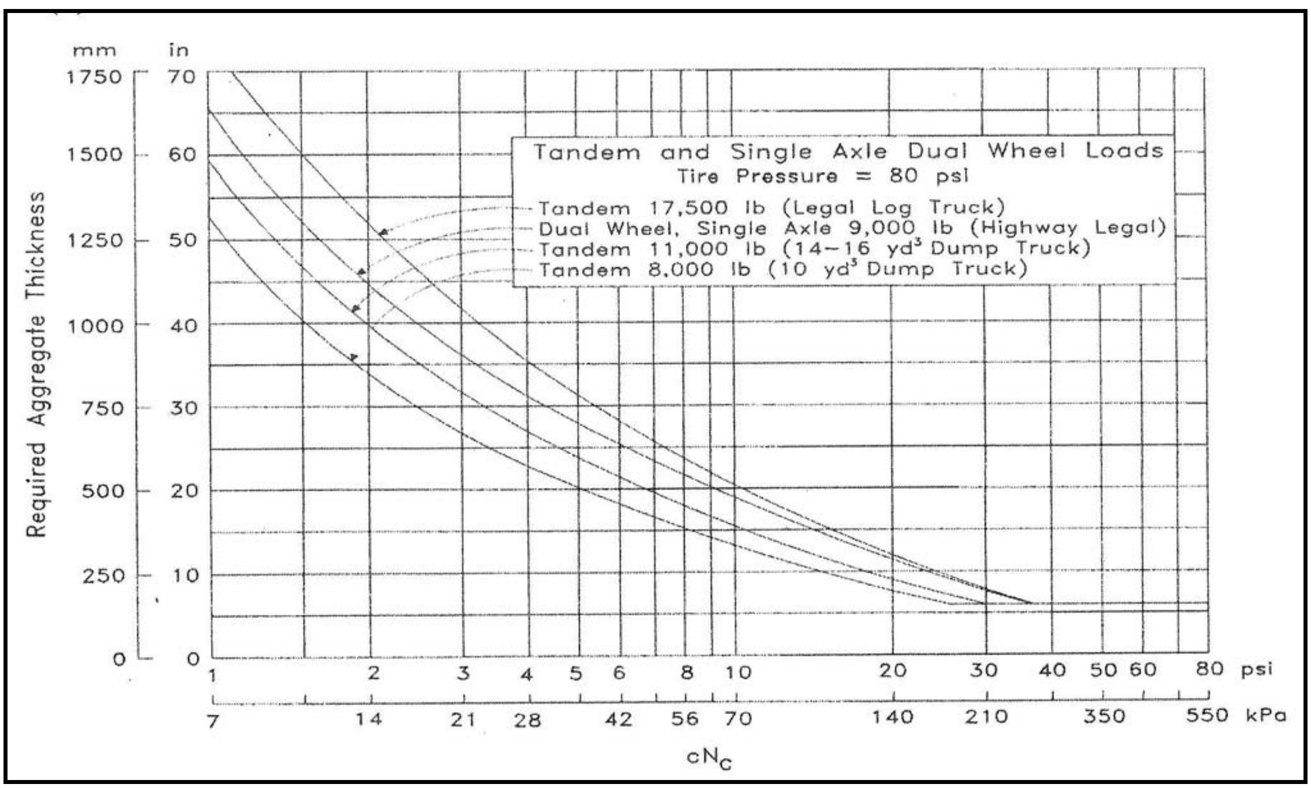


Figure 19-49, USFS Method Bridge Lift Thickness – Dual Wheel Loads (adopted from Geosynthetics Engineering – August 2008)

The seventh step is the selection on the design thickness of the bridge lift as well as material for the bridge lift. The thickness of the bridge lift should be rounded to the next higher thickness divisible by three. The USFS Method is also based on the bridge lift having an in-place CBR of 80, while the stone will obtain this CBR with little effort, the use of granular backfill, having a CBR much lower than 80, requires that the thickness of the bridge lift be increased. Increase the thickness of the bridge lift 3 inches for the use of granular bridge lift materials.

The eighth step is determine the survivability of the geotextile materials for the given soil conditions. Given the anticipated conditions that bridge lifts and reinforcement will be used on, a high survivability is required.

The final step in the USFS Method is developing any plan notes required.

19.10.1.2 Giroud- Han Method

The Giroud-Han method is an iterative process since the required thickness of bridging material is on both sides of the following equation:

$$h = \frac{0.868 + 4\left(0.661 - 1.006J^2\right)\left(\frac{6.3}{h}\right)^{1.5}}{1.816}\left(\sqrt{\frac{80}{\frac{s}{3}\left(1 - 0.9e^{-\left(\frac{6.3}{h}\right)^2}\right)N_c c_u}} - 1\right)6.3 \quad \text{Equation 19-57}$$

Where, $$\left(0.661 - 1.006J^2\right) > 0 \quad \text{Equation 19-58}$$

Where,

h = Required bridge lift thickness (inches)
J = Aperture stability modulus
$\tau = c_u$ = Undrained shear strength (psi)
s = Maximum rut depth (inches)
N_c = Bearing Capacity Factor (see Table 19-30)

Table 19-29, Bearing Capacity Factor and Aperture Stability Modulus (adopted from Geosynthetics Engineering – August 2008)

	Bearing Capacity Factor (N_c)	Aperture Stability Modulus (J[1,2])
Unreinforced	3.14	0
Geotextile Reinforced	5.17	0
Geogrid Reinforced	5.71	J[1,2]

[1]Aperture Stability Modulus determined by geogrid manufacturer/supplier

[2](dimensionless in Equation 19-56, but reported in N-m/degree)

The following assumptions and limitations are placed on the Grioud-Han method.

- Rut depth (s) is limited to 2 to 4 inches
- CBR of stone is greater than 30
- CBR of granular material (A-1 through A-2-6) is greater than 10
- The number Equivalent Single Axle Loads (ESALs) is 10,000
- The tension membrane effect was not taken into account, since it is negligible for rut depths less than 4 inches
- The radius of tire contact (r) is 6.3 inches
- Tire pressure (p) is 80 pounds per square inch
- The wheel load (P) is 10.0 kips
- The minimum thickness of bridge lift is 6 inches

The capacity of the existing subgrade soils should be determined to check whether reinforcement is needed or not using the following equation.

$$P_{h=0,unreinf} = \left(\frac{s}{3}\right)391.53c_u \qquad \text{Equation 19-59}$$

Where,

$P_{h=0,\ unreinf}$ = Unreinforced subgrade support capacity with no bridge lift

If $P_{h=0,\ unreinf}$ is greater than P, no reinforcement is required; however, a 6-inch bridge lift is recommended to prevent disturbance of the existing subgrade. If P is greater than $P_{h=0,\ unreinf}$, then reinforcement is required and Equation 19-56 should be used to determine the required thickness of bridge lift. Utilizing a P of 10,000 pounds in Equation 19-58, the minimum undrained shear strength with corresponding rut depth is shown in the following table.

Table 19-30, Minimum Undrained Shear Strength versus Rut Depth

Rut Depth (inches)	Undrained Shear Strength (c_u, τ) (psf)
2	5520
3	3675
4	2750

19.10.2 Reinforced Embankment

The design approach for the reinforced embankment is to prevent failure. Figure 19-50 provides depictions of potential modes of failure. These potential modes of failure indicate the type of analyses that will be required. In addition, the settlement of the embankment and the creep rate of the reinforcement needs to be considered as well. Creep of the reinforcement only becomes an issue if the creep rate of the reinforcement is greater than the increase in shear strength of the subgrade soils. The most critical condition for embankment stability is at the end of construction. Therefore, a total stress analysis should be performed. A reinforced embankment is different from a Reinforced Soil Slope (RSS – Chapter 18 and Appendix D). An RSS is defined in Chapter 18 as having slopes ranging from 2H:1V to 1H:1V. A reinforced embankment has flatter slopes and up to three layers of reinforcement at the bottom of the embankment.

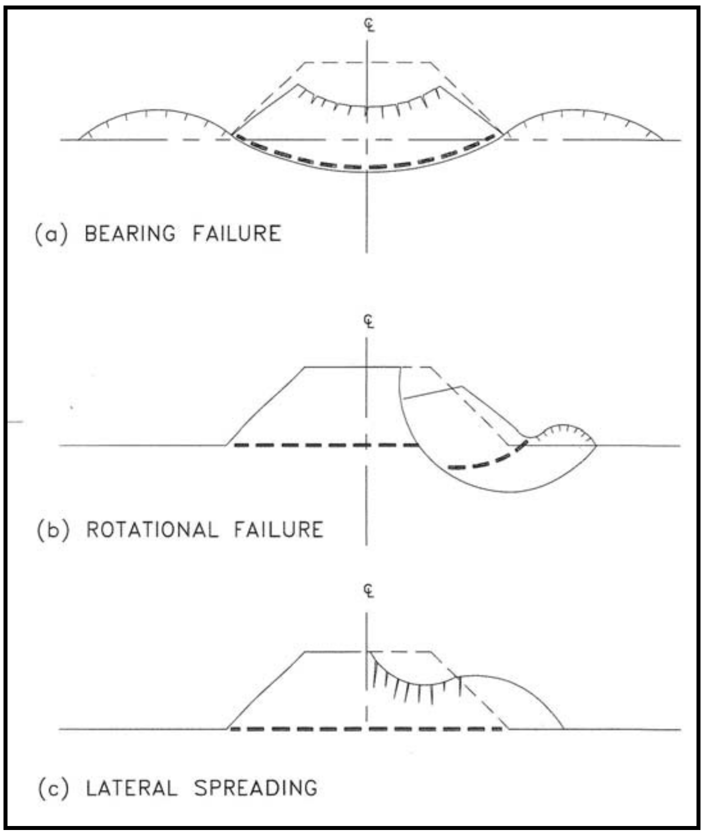


Figure 19-50, Reinforced Embankment Failure Modes (Ground Improvement Methods – August 2006)

19.10.2.1 Reinforced Embankment Design

The following design procedure is adopted from the design steps presented in the Ground Improvement Methods manual.

1. Geometry and Loading Conditions – The geometric parameters required are the height and length of the embankment over the soft foundation soils, the width of the crest (shoulder break-to-shoulder break), and the slide slope angle. The loading conditions include any surcharges and any temporary or dynamic loads. The construction rate

should also be included, because the gain in shear strength is directly affected by the placement of the embankments.

2. Soil Profile and Engineering Properties – The subsurface stratigraphy should be determined, including soil layering and groundwater table location for the foundation soils. The testing should include basic classification testing. The shear strength and consolidation properties should be determined either from correlations with field testing or from laboratory testing. The spatial variation (length and depth) of the soil properties should also be determined.

3. Embankment Fill Engineering Properties – The engineering properties of the fill material should be determined, including basic classification testing (grain-size distribution and moisture-plasticity relationship), moisture-density relationship, shear strength, and chemical properties. The first 18 to 24 inches of fill materials shall consist of free draining granular materials. Above this material, normal backfill materials may be placed.

4. Establish Resistance Factors and Performance Limits – The Resistance Factors and Performance Limits shall meet the requirements contained in Chapters 9 and 10, respectively. The stability analyses performed in the following steps are for the Strength limit state at the end of construction. The end of construction is the most critical condition. Therefore, the Extreme Event will not be checked. The Extreme Event will be checked using the shear strength anticipated from the increase with time (see Chapter 17).

5. Bearing Capacity Check – When the thickness of the soft foundation soil is much greater than the width of the embankment, the following equation may be used to determine the ultimate bearing capacity:

$$q_{ult} = \gamma_{fill} H = c_u N_c \qquad \text{Equation 19-60}$$

Where,

q_{ult} = Ultimate bearing capacity
N_c = 5.14
c_u = Undrained shear strength of foundation soil
γ_{fill} = Unit weight of fill material
H = Height of embankment

If the thickness of the soft soil is less than the width of the embankment, check lateral squeeze using the procedure provided in Chapter 17.

6. Rotational Shear Stability Check – Perform a rotational slip surface analysis (see Chapter 17) on the unreinforced embankment and foundation to determine the critical failure surface and the resistance factor against local shear instability. If the calculated resistance factor is less than required, then, reinforcement is not required. If the resistance factor is greater than required, then, calculate the required reinforcement strength (T_g) to provide an adequate resistance factor using the Figure 19-51 and the following equation:

$$T_g = \frac{\Delta M_R}{y}$$ Equation 19-61

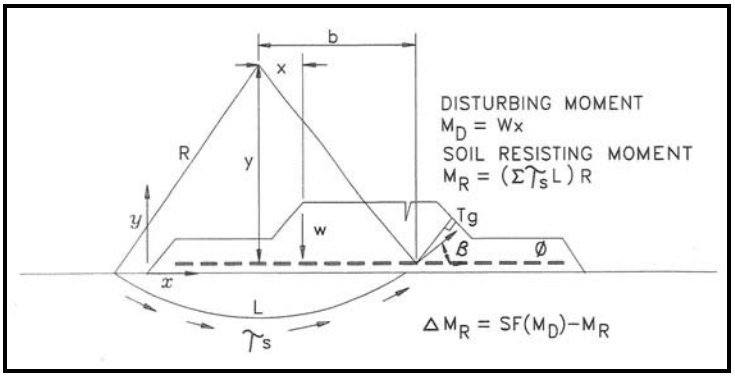


Figure 19-51, Rotational Failure Model
(Ground Improvement Methods – August 2006)

Note: SF = 1/φ

7. Sliding Block Stability Check – Perform a sliding block analysis (see Chapter 17). If the calculated resistance factor is less than required, then, reinforcement is not required. If the resistance factor is inadequate, then, determine the lateral spreading strength of reinforcement (T_{ls}) required (Figure 19-52). The soil/geosynthetic cohesion, C_a, should be assumed to be 0 for extremely soft soils and low embankments. A cohesion value should be included with placement of all subsequent fills in staged embankment construction. In addition to checking for rupture, sliding of the embankment, the sliding of the embankment on top the reinforcement, should be checked (Figure 19-53).

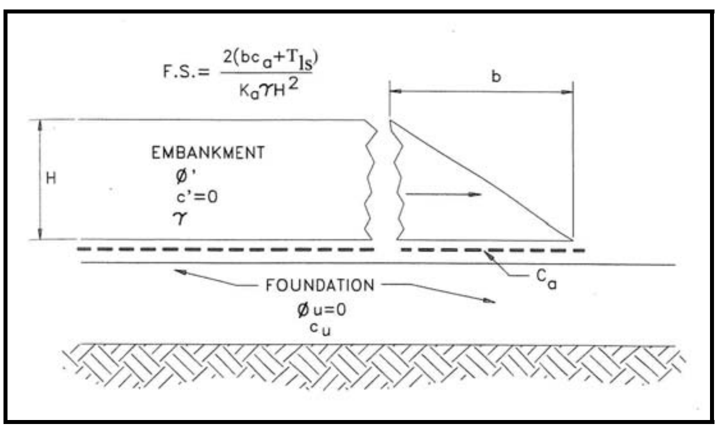


Figure 19-52, Sliding Failure – Rupture of Reinforcement
(Ground Improvement Methods – August 2006)

Note: FS = 1/φ

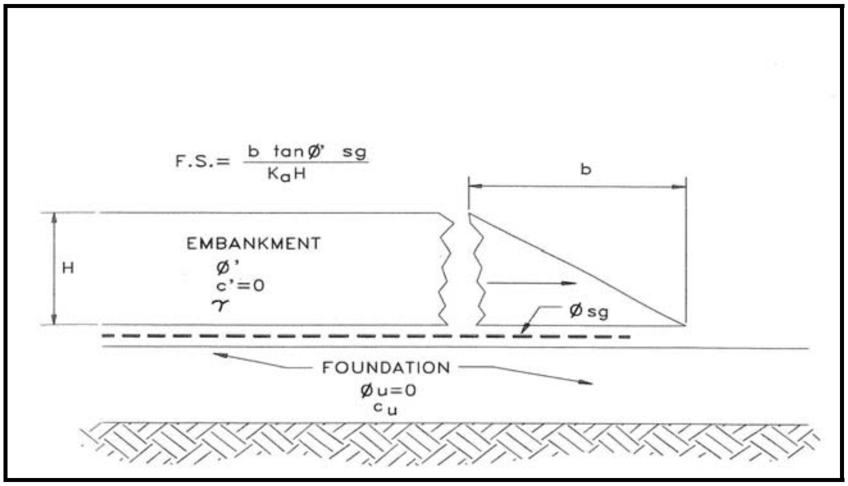


Figure 19-53, Sliding Failure of Embankment over Reinforcement (Ground Improvement Methods – August 2006)

Note: FS = 1/φ

8. Establish Tolerable Geosynthetic Deformation - The deformation of the geosynthetic reinforcement is required to develop the tensile capacity required to prevent failure. The strain in the geosynthetic reinforcement is provided in the following equation.

$$\varepsilon_{geosyn} = \frac{T_{ls}}{J} \qquad \text{Equation 19-62}$$

Where,

ε_{geosyn} = Strain in the geosynthetic

Cohesionless Soils: ε_{geosyn} = 5 to 10 percent

Cohesive Soils: ε_{geosyn} = 2 percent

Peats: ε_{geosyn} = 2 to 10 percent

T_{ls} = Lateral spreading strength of reinforcement

J = Reinforcement Modulus

The maximum strain in the geosynthetic reinforcement will be approximately twice the average strain in the embankment.

9. Establish Geosynthetic Strength Requirements – Most embankments are relatively long and narrow in shape. Thus, during construction, stresses are imposed on the geosynthetic in the longitudinal direction (i.e., along the direction of the centerline). Reinforcement may also be required for loadings that occur at bridge abutments, and due to differential settlements and embankment bending, especially over nonuniform foundation conditions and at the edges of soft soil deposits, because both rotational and sliding block failures are possible in the direction along the alignment of the embankment. This determines the longitudinal strength requirements of the geosynthetic. Because the usual placement of the geosynthetic is in strips perpendicular to the centerline, the longitudinal stability will be controlled by the strength of the transverse seams.

10. Selection of Geosynthetic Reinforcement – Once the geosynthetic strength requirements are established, the geosynthetic reinforcement should be selected that meets the required strength and deformation (strain) requirements.

11. Estimate Magnitude and Rate of Embankment Settlement – The magnitude and rate of embankment settlement should be determined using the procedures outlined in Chapter 17.

12. Establish Construction Sequence and Procedures – The construction sequence and procedures should be established. Proper placement and performance of the geosynthetic is highly influenced by the construction sequence and procedure. The sequence and procedure should be as clear and concise as possible to prevent misunderstandings during construction.

13. Establish Construction Observation Requirements – Since implemented construction procedures are crucial to the success of reinforced embankments on very soft foundations, competent and professional construction inspection is absolutely essential. Field personnel must be properly trained to observe every phase of the construction and to ensure that:

    a. The specified material is delivered to the project
    b. The geosynthetic is not damaged during construction
    c. The specified construction sequence is explicitly followed

    Instrumentation requirements should also be established. As a minimum, install piezometers, settlement points, surface survey points and slope inclinometers. Part of the instrumentation requirements is establishing who will obtain the measurements and how often the measurements will be obtained.

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