Chapter 19

GROUND IMPROVEMENT

GEOTECHNICAL DESIGN MANUAL

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CHAPTER 19

GROUND IMPROVEMENT

19.1 INTRODUCTION

According to Charles (2002), the process of altering the ground is ground treatment, while the purpose of the process is ground improvement, and the result of the process is ground modification. The term "ground improvement" is used generically to mean ground treatment, ground improvement, and ground modification throughout the GDM. The "Preface" to Schaefer, et al. – Volumes I and II (2017) states:

> One of the major functions of geotechnical engineering is to design, implement, and evaluate ground improvement schemes for infrastructure projects. During the last *40* 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 Schaefer, et al. - Volumes I and II (2017), but should not be considered the complete discussion of ground improvement methods. The GEOR should consult each volume as required for more details concerning a specific ground improvement method.

The GEOR should also consult the software package developed by the Strategic Highway Research Program (SHRP2) of the Transportation Research Board (TRB). The software package is called *GeoTech Tools – Geo-Construction Information and Technology Selection Guidance for Geotechnical, Structural & Pavement Engineers*, (GeoTech Tools). GeoTech Tools is located at: http://www.geotechtools.org/ and requires user registration prior to use.

In keeping with the geotechnical philosophy described in Chapter 7, it is incumbent on the GEOR 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 OES/GDS with concurrence from the RPG/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 SSL
- Transfer embankment and/or ERS loads to more competent layers

There are 3 general 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
3. Transfer the load to a more competent (deeper) foundation soil

The “Introduction” to Schaefer, et al. – Volumes I and II (2017), 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 Schaefer, et al. - Volumes I and II (2017))

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 general site conditions including 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	Develop a short-list of geotechnologies applicable to site conditions (Table 19-2 should be used in this selection process)
6	Identify project constraints
7	Identify project risks
8	Preliminary design
9	Identify alternative solutions (i.e., bridge, re-route, deep foundations, etc.)
10	Evaluate project requirements, constraints, and risks against factors affecting geotechnology selection (Tables 19-3 and 19-4)
11	Compare short-list of geotechnology alternatives with geotechnology selection factors
12	Select a preferred geotechnology (see Table 19-5)

Table 19-2, Ground Improvement Categories, Functions and Methods (modified from Schaefer, et al. - Volumes I and II (2017))

Category	Function	Method
Consolidation	Accelerate consolidation and increase shear strength	1.) Prefabricated Vertical Drains (PVDs) 2.) Surcharge
Load Reduction	Reduce load on foundation and reduce settlement	1.) Lightweight fill 2.) Geofoam 3.) Foamed Concrete
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 3.) Stone Columns
Reinforcement	In soft foundation soils, increases shear strength, resistance to liquefaction, and decreases compressibility	1.) Stone Columns 2.) Piles or Drilled Shafts
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.) Deep mixing methods 2.) Mass mixing methods
Grouting	To 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)

Step 10 from Table 19-1 establishes the process for evaluating project requirements against fairly common factors that affect the selection of an appropriate geotechnology for ground improvement. Eighteen Importance Selection Factors (ISFs) have been identified and indicated in Table 19-3. The ISFs are listed in no particular order. Additional factors may be considered based on the requirements of the design team. Each factor is evaluated based on its relevancy and importance to the project requirements and site constraints. Each ISF is assigned an importance rating (IR) from 0, the least important, to 3, the most important. Table 19-4 depicts an example table of the ISFs and IR for each factor.

Table 19-3, Ground Improvement Importance Selection Factors (ISF) (modified from Schaefer, et al. - Volumes I and II (2017))

Speed of construction	Familiarity with geotechnology
Minimize construction disturbance	Design procedure
Longevity of constructed works	Contracting
Cost of construction	Life-cycle cost
Constructability	Project constraint – construction season
ROW requirements or restrictions	Additional project constraint
Aesthetics	Project risk – delay due to settlement time
Environmental	Project risk – quality assurance
Degree of establishment	Additional project risk

Table 19-4, Weighted Geotechnology Selection Factors

ISF[1]	IR[2]
Speed of Construction	
Minimize Construction Disturbance	
Longevity of Constructed Works	
Cost of Construction	
Constructability	
ROW Requirements or Restrictions	
Aesthetics	
Environmental Concerns	
Degree of Establishment	
Familiarity with Geotechnology	
Design Procedure	
Contracting	
Life-cycle Cost	
Project Constraint – Construction Season	
Additional Project Constraint (if required)	
Project Risk – Delay Due to Settlement Time	
Project Risk – Quality Assurance	
Addition Project Risk (if required)	

[1]Importance selection factor (ISF)
[2]Importance rating of each Geotechnology selection factor based on project requirements and site constraints. Each factor should be rated between 0, least importance factor, and 3, most important factor.

The final step in selecting an appropriate geotechnology for ground improvement is to determine the most acceptable type. Each ISF is assigned a suitability factor (SF). The SF is based on how suitable a particular geotechnology will achieve the required ground improvement considering the ISF and the importance of each ISF. SF ranges from 4, most suitable, to 1, least suitable. The determination of SF is very subjective; every effort should be made to avoid making a specific type of geotechnology appear suitable. Any cost associated with a selection factor should be considered when developing the rating. This determination is made based on the IR and SF for each ISF. A weighted rating (WR) is developed as the product of IR and SF. A total weighted rating (WR_T) is determined (see Equation 19-1). The geotechnology with the highest WR_T should be selected for the specific project site. Other highly scored geotechnologies may be included in the Contract Documents as acceptable alternatives. Table 19-5 provides an example of this process. The Geotechnology Selection Matrix is available on the Geotechnical page of the SCDOT website; https://www.scdot.org/business/geotech.aspx.

$$WR_T = \sum_{i=1}^{n}(IR_i * SF_i) = \sum_{i=1}^{n} WR_i \qquad \textbf{Equation 19-1}$$

Table 19-5, Geotechnology Selection Matrix

ISF \ Geotechnology	IR	Geotechnology A[1] SF	Geotechnology A[1] WR	Geotechnology B[1] SF	Geotechnology B[1] WR	Geotechnology C[1] SF	Geotechnology C[1] WR
Speed of Construction	3	4	**12**	3	**9**	4	**12**
Minimize Construction Disturbance	2	1	**2**	1	**2**	1	**2**
Longevity of Constructed Works	2	3	**6**	3	**6**	1	**2**
Cost of Construction	2	3	**9**	1	**3**	1	**3**
Constructability	1	1	**1**	4	**4**	1	**1**
ROW Requirements or Restrictions	0	4	**0**	1	**0**	4	**0**
Aesthetics	2	4	**8**	4	**8**	2	**4**
Environmental Concerns	2	3	**6**	4	**8**	4	**8**
Degree of Establishment	3	4	**12**	1	**3**	3	**9**
Familiarity with Geotechnology	1	2	**2**	3	**3**	4	**4**
Design Procedure	2	4	**8**	3	**6**	2	**4**
Contracting	2	2	**4**	2	**4**	3	**6**
Life-cycle Cost	1	2	**2**	3	**3**	1	**1**
Project Constraint – Construction Season	0	1	**0**	1	**0**	1	**0**
Additional Project Constraint (if required)	0	1	**0**	1	**0**	1	**0**
Project Risk – Delay Due to Settlement Time	2	4	**8**	3	**6**	4	**8**
Project Risk – Quality Assurance	3	3	**9**	1	**3**	1	**3**
Addition Project Risk (if required)	0	2	**0**	4	**0**	2	**0**
Total Weighted Rating (WR_T)			**89**		**57**		**67**

[1]SF for each geotechnology are based on project requirements and site constraints. Each SF should be rated between 1, least suitable, and 4, most suitable.

Schaefer, et al. – Volumes I and II (2017) also indicate pavement support stabilization as well as reinforced soil structures as ground improvement categories. Pavement support stabilization will not be discussed in the GDM. In addition, the use of reinforced soil structures (i.e., RSSs and MSE walls) is discussed in Chapters 17 and 18, respectively, and therefore, will not be discussed in this Chapter.

The cost of the ground improvement geotechnology must be considered in the selection process. Contact the OES/GDS for cost information for ground improvement methods previously used by SCDOT. For ground improvement geotechnologies not previously used, every effort should be made to contact at least 3 contractors to obtain approximate pricing information.

According to the "Introduction" to Schaefer, et al. – Volumes I and II (2017):

> For many years the term QC/QA was used to describe quality activities associated with construction where Quality Control (QC) referred to the quality activities conducted by the contractor and Quality Assurance (QA) referred to quality activities conducted by the owner. More recently, the term Quality Assurance is being used as the umbrella term that includes the contractor's QC activities and the acceptance functions of the owner-agency. AASHTO (2006), FHWA (2008) and Code of Federal Regulations (23 CFR 637) all define the core elements of a construction Quality Assurance Program to include:
>
> 1. Contractor Quality Control (QC)
> 2. Agency Acceptance
> 3. Agency Independent Assurance (IA)
> 4. Dispute Resolution
> 5. Laboratory Accreditation and Qualification
> 6. Personnel Qualification/Certification
>
> All 6 elements are deemed necessary to have a complete and effective QA Program. A QA program missing any one or more of the elements is not sufficient and should not be construed as being "substantially complete" with the intent of the AASHTO guidelines or the federal regulation.

The costs for the Quality Assurance Program needs to be added to the total cost of the soil and site improvement method.

19.2 PREFABRICATED VERTICAL DRAINS

Prefabricated vertical drains (PVDs), also commonly called wick drains, are used to accelerate consolidation of compressible Clay-Like 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 Clay-Like soils under a pressure gradient induced by excess pore pressures associated with the placement of permanent fill and/or surcharge fill (see Figure 19-1).

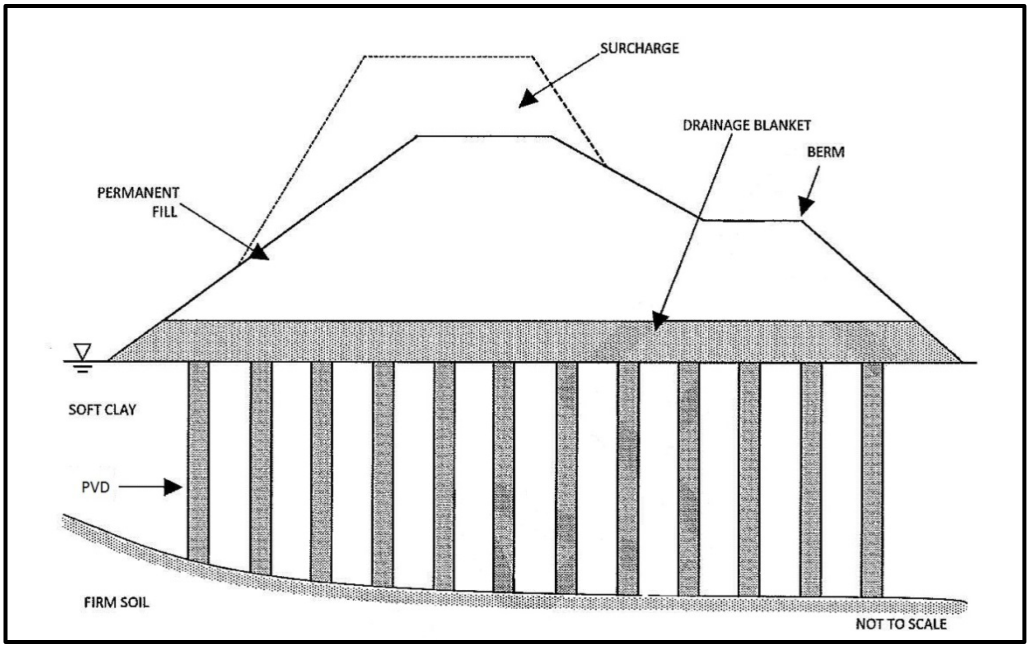


Figure 19-1, PVD Installation for a Highway Embankment (modified Schaefer, et al. – Vol. I (2017))

PVDs have numerous advantages some of which include economy, installation speed, continuity of drain, and minimal displacement. Additional advantages are presented in Schaefer, et al. – Vol. I (2017), 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)
- Installation problems through dense subsurface obstructions

PVDs are thin strips (about 1/8 inch thick by 4 inches wide) consisting of a rigid core sheathed in filter fabric. PVDs have generally replaced sand drains. However, the PVD 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 can also provide vertical drainage and similar methods can be applied to evaluate their effect on settlement rates.

19.2.1 Design Concepts

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 should 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 to penetrate dense materials and obstructions. 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 Schaefer, et al. – Vol. I (2017), "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."

$$t = \frac{D^2}{8c_h}(F(n) + F_s + F_r)\,ln\left(\frac{1}{1-\overline{U_h}}\right) \qquad \text{Equation 19-2}$$

$$F(n) = ln\left(\frac{D}{d}\right) - 0.75 \qquad \text{Equation 19-3}$$

Where,

t = Time required to achieve desired average degree of consolidation
$\overline{U_h}$ = 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 19-3)
d = Equivalent circular drain diameter
F_s = Factor for soil disturbance
F_r = Factor for well resistance

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. 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, and 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 $F_s = 0$. Soil 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 F_r

The well resistance factor is normally assumed to be negligible (i.e., $F_r = 0$), provided the PVD has sufficient discharge capacity. The well resistance is only a factor for very deep PVDs (i.e., greater 150 feet deep). For very deep PVDs, please refer to Chu, Bo, and Choa (2004) for guidance in determining F_r.

19.2.1.3 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 as much as 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-Like soil 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 CPTu instrumentation and allowing for pore pressure dissipation.

19.2.1.4 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. According to Chu, et al. (2004), a mandrel having a rhombic shape as shown in Figure 19-2 causes the least disturbance on the in-situ soils during installation of the PVD. Alternatively, d may be determined using the following equation:

$$\boldsymbol{d} = \frac{[\boldsymbol{2*(a+b)}]}{\boldsymbol{\pi}}$$ **Equation 19-4**

Where,

a = Width of PVD, inches (see Figure 19-2)
b = Thickness of PVD, inches (see Figure 19-2)

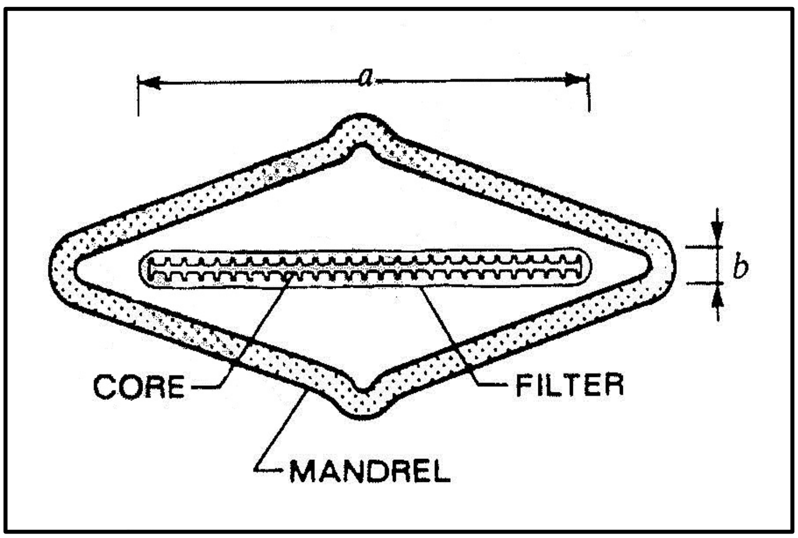


Figure 19-2, PVD Dimensions
(modified Schaefer, et al. – Vol. I (2017))

19.2.1.5 Determination of $\overline{U_h}$

The average degree of consolidation ($\overline{U_h}$) should develop the required settlement to 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.

19.2.1.6 Determination of D

According to Schaefer, et al. – Vol. I (2017):

> 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.

19.2.1.7 Determination of t

The time (t) is the duration required to achieve the desired $\overline{\boldsymbol{U_h}}$ for a cylinder of diameter (D) and drain diameter (d). According to Schaefer, et al. – Vol. I (2017), "There are *3* basic variables that can be manipulated in order to achieve a desired result from Equation *19-2*. 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.

19.2.1.8 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 2.0, 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 should 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 (as required in Chapter 26) 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 an electronic copy of 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.
12042 SE Sunnyside Road, Suite 711
Clackamas, Oregon 97015 USA
Tel. (971) 224-4187

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 relatively thin 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 SCDOT typically uses 4 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 by Pestana, Hunt, and Goughnour (1997). Because of the uncertainty in how the settlements are determined in FEQDrain and based on field experiment test results (see Rollins, Anderson, McCain and Goughnour (2003)), settlements shall be assumed to reduce to approximately 60 percent of the unmitigated settlement instead of those determined by FEQDrain.

EQ drains work by reducing the pore pressure ratio (r_u), 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 anticipated from earthquakes in South Carolina, the r_u shall be limited to 0.50. Using a r_u of this magnitude will cause the drain spacing to become smaller and potentially increasing the drain size.

$$r_u = \frac{\Delta u}{\sigma'_v} \qquad \textbf{Equation 19-5}$$

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 PVDs is shown in Figure 19-3. The contractor is required to submit an installation plan, shop drawings, material samples, and anchorage details. A minimum 12-inch thick layer of drainage material 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.

PVDs shall conform to the requirements of STS SC-M-801-1 (latest version) for *Prefabricated Vertical Drain with Fabric*. The drainage material shall conform to the requirements of Supplement Specification *Bridge Lift Materials* (latest version) and shall consist of stone, granular, or man-made (i.e., lightweight) bridge lift materials. The use of borrow excavation materials as the drainage material is not allowed.

EQ drains are installed in a manner similar to PVDs. The EQ drains shall conform to the requirements of STS SC-M-205-1 (latest version) for *Earthquake Drains*. Similar to PVDs the drainage materials used for EQ drains shall conform to the requirements of Supplemental Specifications *Bridge Lift Materials*, latest version, and shall consist of stone, granular, or man-made (i.e., lightweight) bridge lift materials. The use of borrow excavation materials as the drainage material is typically not sufficient.

The latest version of the Supplemental Specifications and STSs are available on the SCDOT website:

> https://www.scdot.org/business/business-landing.aspx.

In addition, “go-by” drawings are available to assist the GEOR. The GEOR is reminded that the provided “go-by” must be modified for the specific project. The latest version of the “go-by” is available on the SCDOT website:

> https://www.scdot.org/business/geotech.aspx.

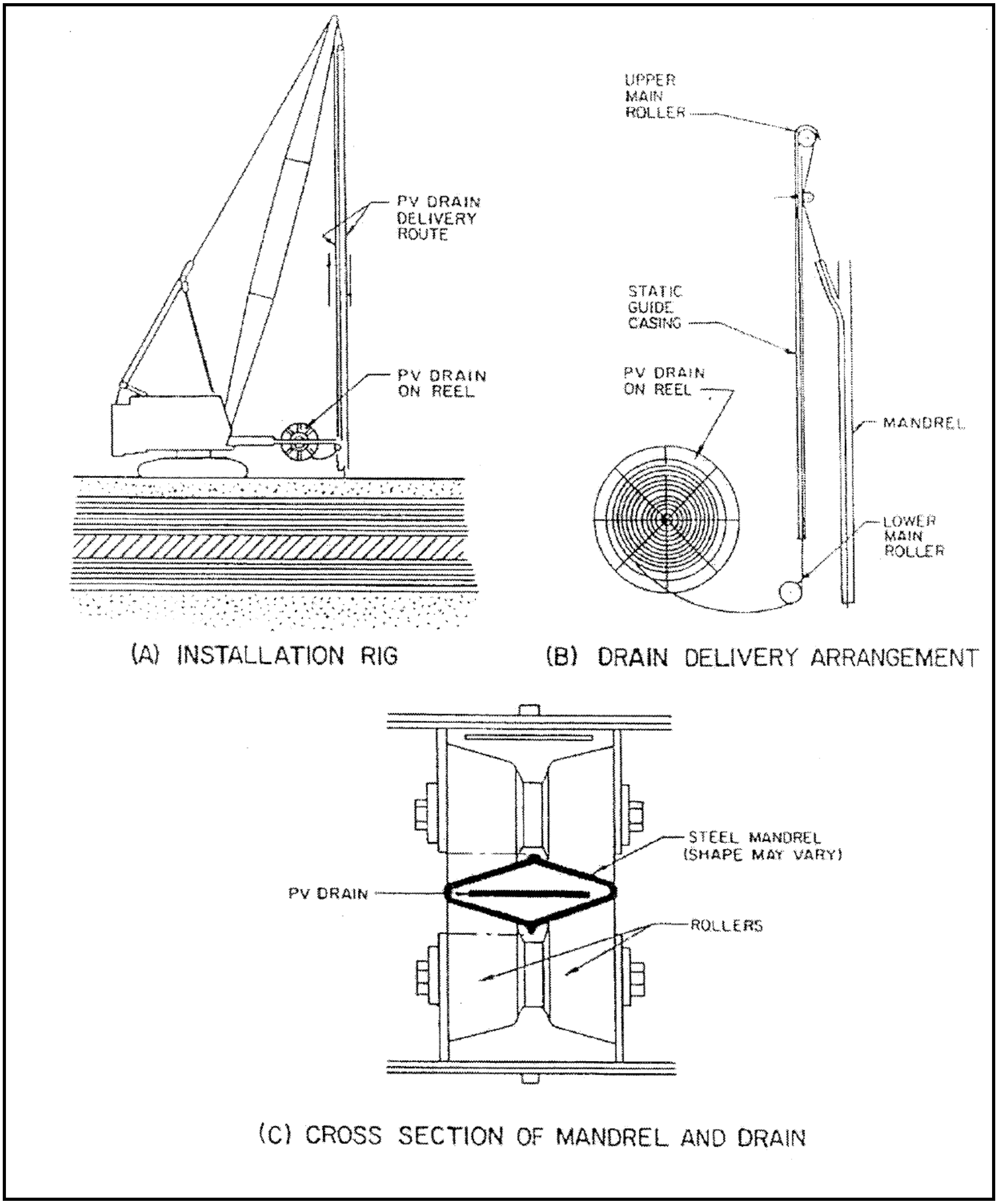


Figure 19-3, Crane Mounted Installation Rig
(Schaefer, et al. – Vol. I (2017))

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 (expanded polystyrene (EPS)) to 65 pcf for expanded shale, clay, and slate (ESCS). Geofoam and lightweight cellular concrete will typically behave like materials that have an inherent compressive strength similar to Clay-Like soils in undrained loading. ESCS and glass aggregate will typically behave and have properties similar to Sand-Like soils. In addition to reducing settlement and increasing stability, lightweight fill materials reduce the load applied to ERSs and increase an embankment's resistance to seismic loads by reducing the seismic inertial forces. Table 19-6 provides a list of lightweight fill materials used by SCDOT. Schaefer, et al. – Vol. I (2017) provides additional lightweight materials; however, the use of these other lightweight fill materials (i.e., wood fiber, blast furnace slag, fly ash, shredded tires, or boiler slag), must be approved in writing by SCDOT (including the RPG/GDS and the Office of Materials and Research (OMR)). Lightweight fill materials shall conform to the requirements of STS SC-M-203-5 (latest version) for *Lightweight Aggregates*. If other lightweight materials have been approved for use, the GEOR is required to prepare a Special Provision for that material. The latest version of the STS is available on the SCDOT website:

https://www.scdot.org/business/business-landing.aspx.

Table 19-6, Lightweight Fill Materials (modified from Schaefer, et al. – Vol. I (2017))

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
Glass Aggregate	9.50 – 12.50	0.15 – 0.20

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 Schaefer, et al. – Vol. I (2017):

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

According to Schaefer, et al. – Vol. I (2017):

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 Schaefer, et al. – Vol. I (2017):

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 and glass aggregate 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 Schaefer, et al. – Vol. I (2017):

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

19.3.1.7 Seismic Considerations

According to Schaefer, et al. – Vol. I (2017):

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 Schaefer, et al. – Vol. I (2017):

- *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 <u>Geofoam</u>

According to Schaefer, et al. – Vol. I (2017), “Geofoam is a generic term used to describe any cellular material used as a lightweight fill, *such as* block molded expanded polystyrene (EPS) and extruded polystyrene (XPS), both plant manufactured.” Geofoam materials have the advantage of being not only lightweight, but also may be cut to any size or shape to fit the requirements of the project. Stark, Arellano, Horvarth and Leschinsky (2004), “Guideline and Recommended Standard for Geofoam Applications in Highway Embankments”, contains detailed design guidelines for the use of EPS geofoam in roadway and bridge embankments. Geofoam is a lightweight fill material that has a specific compressive strength.

According to Schaefer, et al. – Vol. I (2017):

> The overall design process when using EPS geofoam is divided into *3* phases in order to consider the interaction between the *3* major components in the embankment.
>
> 1. Design to preclude external (global) *instability* of the embankment. This should include considerations for settlement, bearing capacity, and slope stability*/instability* under the projected loading conditions.
> 2. 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.
> 3. Design of an appropriate pavement system for the subgrade provided by the underlying geofoam blocks.
>
> Stability analyses require the modeling and quantifying of both internal shear strength of the geofoam and the shear strength of the geofoam interfaces. The internal shear strength of EPS geofoam correlates to its compressive strength. The interfaces typically include geofoam to geofoam, geofoam to soils and geofoam to geosynthetic material. Interface friction is an important stability design consideration, particularly under horizontal wind, water, and/or seismic loading conditions.
>
> The range of densities and compressive resistance for *Rigid Cellular Polystyrene (RCPS)* Geofoam are listed in ASTM D6817 - *Standard Specification for Rigid Cellular Polystyrene Geofoam*. There are *7* grades of EPS that range in density from 0.70 to 2.85 *pounds per cubic foot (pcf)*, with compressive resistance values of 2.2 to 18.6 *pounds per square inch (psi) at 1 percent strain*, respectively. Six grades of XPS are listed *in ASTM D6817. Table 19-7 provides the density; compressive resistance at 1, 5, and 10 percent strains; and the flexural strength as described in ASTM D6817. The latest version of ASTM D6817 should be consulted to ascertain the relevant properties of the geofoam*. Densities and compressive strengths range from 1.2 to 3.0 pcf and 2.9 to 40.6 psi.
>
> Geofoam embankments often support an overlying roadway pavement. The objective in the design of an appropriate pavement system is to select the most economical arrangement and thickness of pavement materials for the subgrade provided by the supporting EPS blocks. Equivalent soil subgrade strengths for the EPS blocks can be used with traditional pavement design procedures. Subgrade properties as a function of EPS block density are listed in *Table 19-8*.

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.

Table 19-7, Physical Properties of RCPS Geofoam (modified from ASTM (2015))

Material Designation	Density (pcf)	Compressive Resistance (psi)			Flexural Strength (psi)
		1% Strain	5% Strain	10% Strain	
EPS12	0.70	2.2	5.1	5.8	10.0
EPS15	0.90	3.6	8.0	10.2	25.0
EPS19	1.15	5.8	13.1	16.0	30.0
EPS22	1.35	7.3	16.7	19.6	35.0
EPS29	1.80	10.9	24.7	29.0	50.0
EPS39	2.40	15.0	35.0	40.0	60.0
EPS46	2.85	18.6	43.5	50.0	75.0
XPS20	1.20	2.9	12.3	15.0	40.0
XPS21	1.30	5.1	16.0	15.0	40.0
XPS26	1.60	10.9	26.8	25.0	50.0
XPS29	1.80	15.2	34.1	40.0	60.0
XPS36	2.20	23.2	46.6	60.0	75.0
XPS48	3.00	40.6	77.6	100.0	100.0

Table 19-8, Equivalent Soil Subgrade Values of EPS Geofoam (modified from Schaefer, et al. – Vol. I (2017))

EPS Block Density (pcf)	CBR (%)	Young's Modulus (psi)	Resilient Modulus (psi)
1.25	2	725	725
1.5	3	1015	1015
2.0	4	1450	1450

Table 19-9 summarizes the design parameters associated with the use of EPS geofoam.

Table 19-9, EPS Geofoam Design Guidelines (modified from Schaefer, et al. – Vol. I (2017))

<table>
<tr><th colspan="4">Design Parameters</th></tr>
<tr><td>Density, dry</td><td>0.75 – 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 φ = 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 <u>Foamed Concrete</u>

Foamed concrete (lightweight cellular concrete) is created by introducing preformed foam into cement water slurry. The preformed foam is designed for concrete and creates a network of discrete air cells within the cement/water 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 Schaefer, et al. – Vol. I (2017)):

> *Lightweight cellular* concrete (a.k.a. foamed concrete) is a liquid product that is practically self-leveling, and can be pumped over a distance as great as 3,300 feet. The *lightweight cellular* concrete will begin to harden between 2 to 6 hours after placement, and generally solidifies in 24 hours. Design with this product is analogous to design with conventional concrete. The maximum cast unit weight and related minimum compressive strength should be specified as dictated by design and with considerations of local suppliers of *lightweight cellular* concrete.

The range of wet cast density and compressive strength that can be specified generally can range from 24 to 80 pcf and 10 to 300 psi, respectively.

Table 19-10 summarizes key design considerations.

Table 19-10, Foamed Concrete Design Guidelines
(modified from Schaefer, et al. – Vol. I (2017))

<table>
<tr><th colspan="4">Design Parameters</th></tr>
<tr><td>Wet Density Range</td><td>24 - 80 pcf</td><td>Freeze-thaw Resistance, 100 cycles</td><td>92 – 98 %[1]</td></tr>
<tr><td>Compressive Strength</td><td>10 - 300 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">Dry density values will be lower than wet density values.
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

19.3.4 Expanded Shale, Clay & 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 Sand-Like soils. ESCS is a synthetic aggregate created from heating certain shales, clays, and slates in a rotary kiln to temperatures ranging from 1,800° F to 2,200° 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 19-11 summarizes key design considerations.

Table 19-11, ESCS Design Guidelines
(modified from Schaefer, et al. – Vol. I (2017))

<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° (assumes the soil is placed loose).</td></tr>
</table>

19.3.5 Glass Aggregate

Glass aggregate is made from 99 percent recycled glass with a foaming agent added prior to the aggregate being baked in a kiln at approximately 1,650° F. The first step in the manufacturing process is crushing the glass into small pieces and then grinding the small pieces into a powder. A foaming agent is added to the glass powder. At the temperature previously indicated the blend of foaming agent and glass powder melts forming a solid sheet or “cake”. The cake expands as it is heated in the kiln. Bubbles of gas form inside the cake during the heating and follow a torturous path toward the surface of the cake. As the cake cools naturally to room temperature the cake cracks into glass fragments which are then sieved to form glass aggregate.

The design procedures using glass aggregate use conventional geotechnical methods associated with granular soils. Table 19-12 summarizes key design considerations.

Table 19-12, Glass Aggregate Design Guidelines

<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° (assumes the soil is placed loose).</td></tr>
</table>

19.4 AGGREGATE COLUMNS

Aggregate (i.e., stone) columns are constructed using a vibratory probe to increase the density of loose sands at depths beyond which surface compaction equipment is inadequate by using stone as a replacement/displacement material. The vibrations in the immediate vicinity of the vibrator induce liquefaction of saturated loose Sand-Like soils. The vibrator densifies loose Sand-Like soils as well as allowing for the insertion of stone into matrix through displacement of the in-situ materials. The mechanical vibrations and water to overcome the in-situ effective stresses between the soil grains allowing the sand grains to rearrange under the action of gravity into a denser state as well as be displaced by the stone. Included in this Section along with stone columns are vibro-concrete columns (VCCs), geotextile-encased columns (GECs), and Geopier® Rammed Aggregate Pier™ (Geopiers). Stone columns are constructed using either vibro-replacement or vibro-displacement. Table 19-13 provides definitions for both terms. Stone columns shall conform to the requirements of STS SC-M-205-2 (latest version) for *Stone Columns*. Prior to specifying the use of Geopiers or VCCs the GEOR shall obtain the acceptance of both the OES/GDS and the RPG/GDS. If Geopiers or VCCs have been approved for use, the GEOR is required to prepare a Special Provision for the Geopier or the VCC. The latest version of the Stone Column STS is available on the SCDOT website:

https://www.scdot.org/business/business-landing.aspx.

Table 19-13, Vibro-replacement and Vibro–displacement Definitions (modified Schaefer, et al. – Vol. I (2017))

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 19-4.

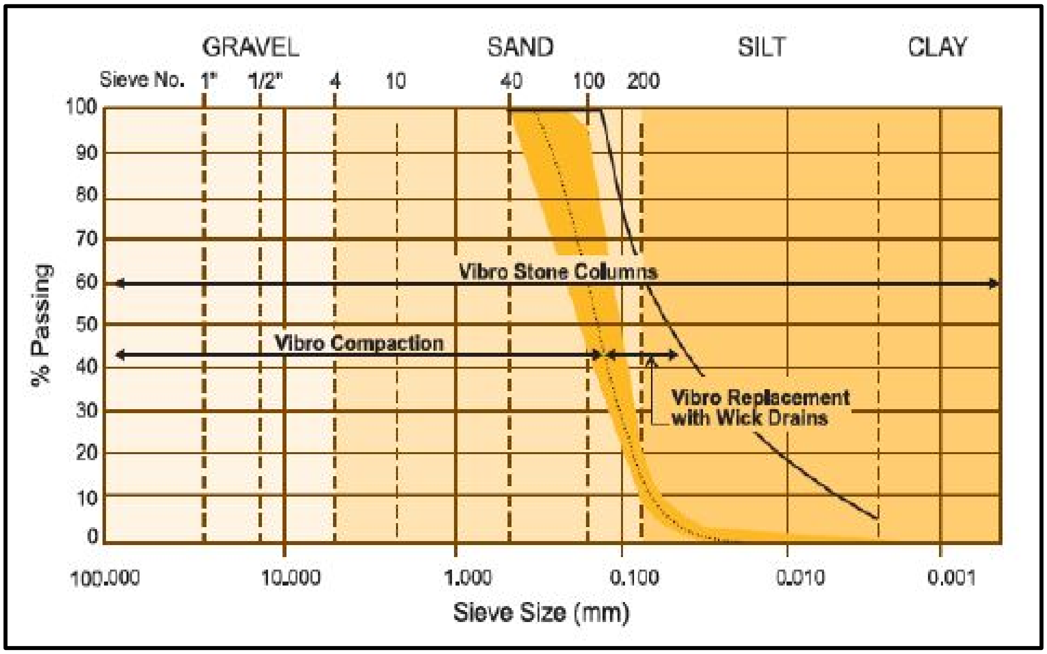


Figure 19-4, Applicable Grain-Size Distributions for Stone Columns (Schaefer, et al. – Vol. I (2017))

19.4.1 Vibro-Replacement

The top feed method is a wet method and replaces the in-situ soil (i.e., vibro-replacement) with the stone column (see Figure 19-5). 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.

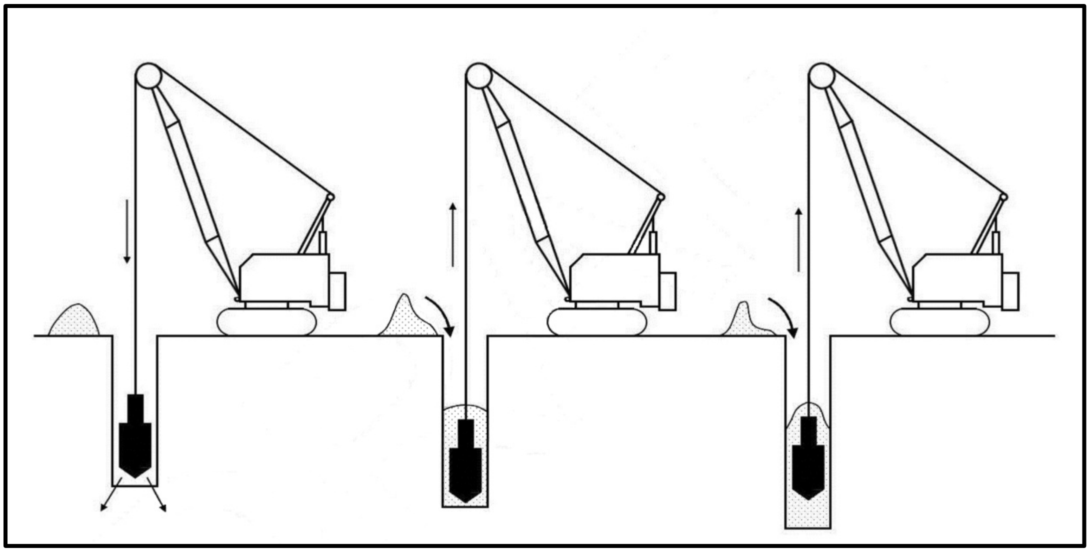


Figure 19-5, Top Feed Construction Method
(Schaefer, et al. – Vol. I (2017))

19.4.2 Vibro-Displacement

When environmental impacts are anticipated, stone columns should be constructed using the vibro-displacement method (see Figure 19-6). 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.

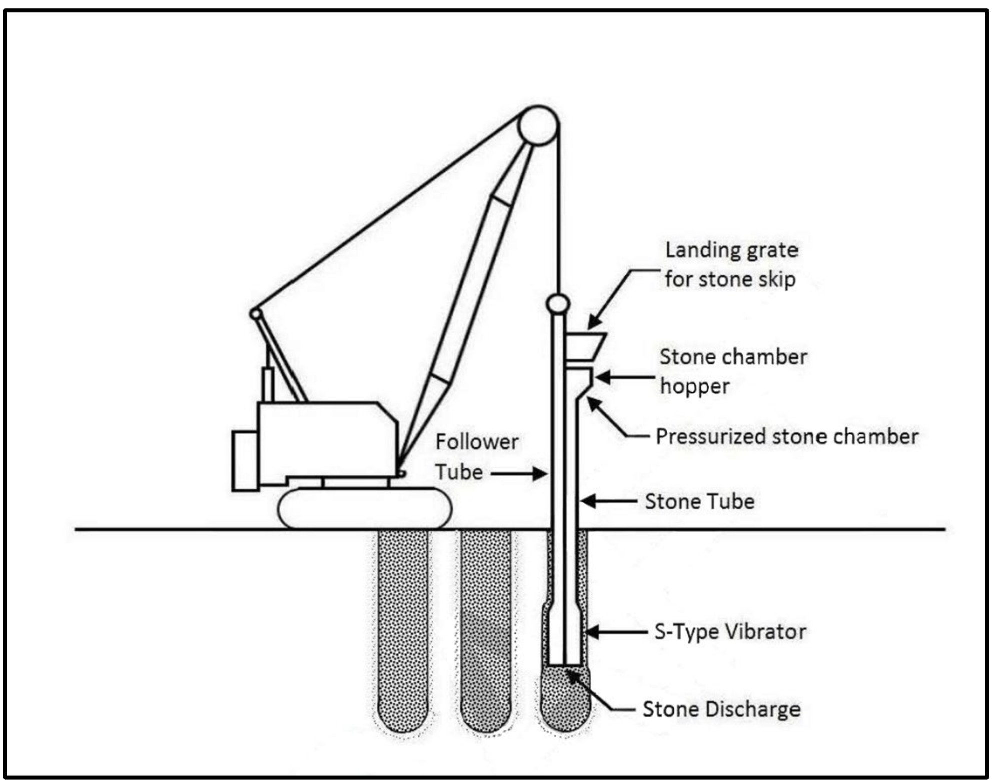


Figure 19-6, Bottom Feed Construction Method
(Schaefer, et al. – Vol. I (2017))

19.4.3 Vibro-Concrete Columns

According to Schaefer, et al. – Vol. I (2017)):

> 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. *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 *VCC* 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 *VCC* (*see Figure 19-7*).

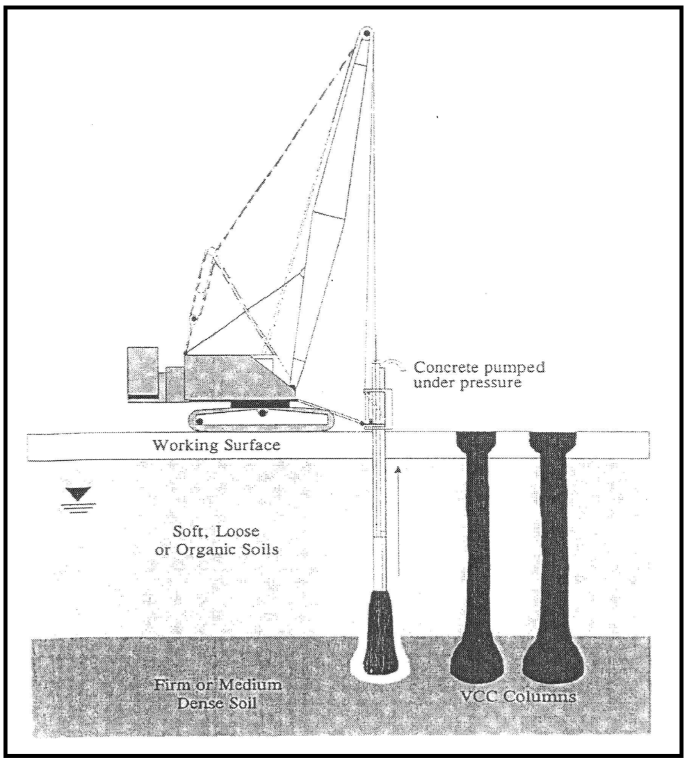


Figure 19-7, Vibro-Concrete Column
(Elias, et al. – Vol. I (2006))

19.4.4 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 a tube is inserted into the void and then the tube is filled with sand or fine gravel. The displacement method uses a steel pipe with 2 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. For additional information about GECs please see GeoTech Tools at: http://www.geotechtools.org/.

19.4.5 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 19-8). Geopiers typically extend to depths of 7 to 35 feet.

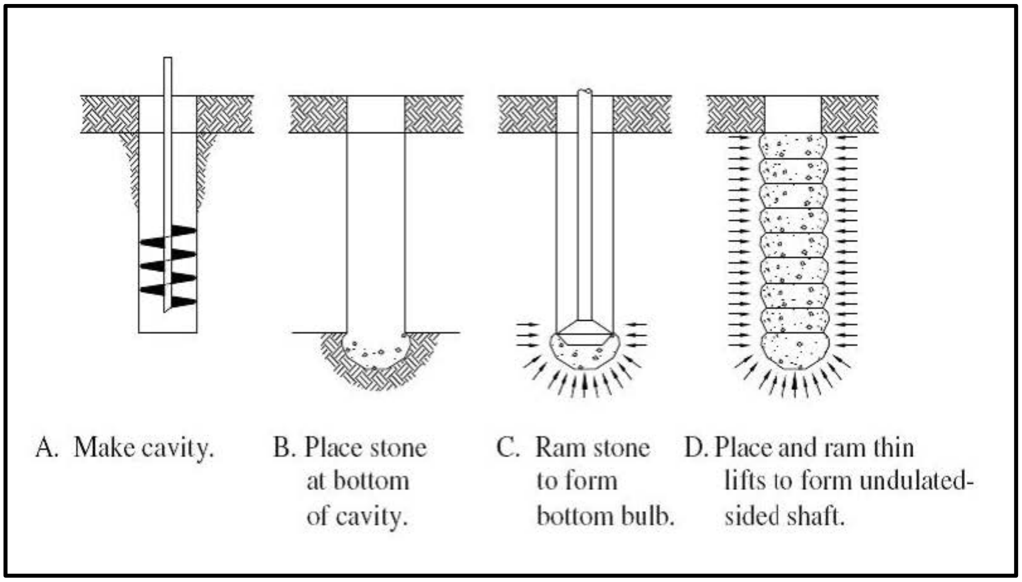


Figure 19-8, Geopier® Rammed Aggregate Pier™ (Geotech Tools (2012))

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 the need for external support (i.e., casing). Shallow groundwater may require the use of temporary casing; however, a specialist contractor should be consulted prior to designing Geopiers with a temporary 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. The tamper should have an area at least 85 percent of the area of the pre-bored hole. This action increases the lateral stress in the surrounding soil, further stiffening the stabilized composite soil mass.

19.4.6 General Considerations

Stone columns can be used to improve the stability of slopes, increase bearing capacity, reduce total and differential settlements, 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 amount 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.

Some of the advantages of stone columns are economy and the 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 other ground improvement options. 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 soil and site 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 Elias, et al. – Vol. I (2006)):

> 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.

19.4.7 Feasibility

According to Schaefer, et al. – Vol. I (2017)):

> 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, aggregate column area, column spacing, and energy applied. Experience has shown that soils with less than 15 percent passing the #200 *(<0.074 mm)* sieve, and clay contents less than 2 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/or displaced by the aggregate columns. In the case of clayey soils, the ground improvement is achieved by reinforcing the soil.
>
> 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 110 kips per column, provided sufficient lateral support by the in-situ soil can be developed.
>
> 2. The most significant improvement is likely to be obtained in compressible *Clay-Like soils* ranging in shear strength from 300 to 1000 psf.
>
> 3. Aggregate 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 300 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 1 to 2 stone column diameters, the ability to develop consistent column diameters becomes questionable.
>
> 4. Ground improvement with stone columns reduce settlements typically from 50 to 70 percent of the unimproved ground response and differential settlement from 5 to 15 percent of unimproved soil response. Ground improvement with rammed aggregate piers can reduce settlement to less than 1 inch.
>
> 5. Due to the development of excessive resistance to penetration of the vibrator a practical upper limit is in the range of undrained shear strength of 1,000 to 2,000 psf for stone columns. If stone columns are used in these stiff soils or through stiff lenses, the column hole is commonly pre-bored, which is often the case in landslide projects. This will result in significant additional cost.
>
> 6. The installation of rammed aggregate piers in soils that do not stand open during drilling (loose *Sand-Like* soils, very soft *Clay-Like* soils) often

requires the use to temporary casing, which reduces the installation rate and increases the cost of the piers.

7. The ultimate capacity of a group of aggregate columns is predicted by estimating the ultimate capacity of a single column and multiplying that capacity by the number of columns in the group.

8. The maximum practical depth of stone columns and rammed aggregate piers is 100 feet and 35 feet, respectively.

9. *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.*

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

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

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*

VCCs use the load transferring characteristics of piles, while mobilizing the full ground improvement potential of aggregate columns. In *Sand-Like* soils, VCCs also densify the surrounding soils by the displacement process, in *Clay-Like* soil this densification does not occur. Construction of the columns is a quiet process, with minimal surface vibration, allowing work close to structures. As VCC installation involves a dry process, limited spoil removal is required. Due to enlarged-base construction with VCCs, column lengths are shorter than would be required for conventional piles. Where thick strata of very soft clay or organic material such as peat are present, they can also be technically feasible and economic solution.

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

1. The allowable design load for VCCs 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 150 to 250 kips.

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

3. Typical *VCC* lengths vary from 20 to 75 feet.

19.4.8 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 water 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.

19.4.9 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, Barksdale and Bachus, (1983)
2. "The Design of Vibro Replacement," Priebe, (1995)
3. See Aggregate Columns on GeoTech Tools at: http://www.geotechtools.org/

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, and the area replacement and stress ratios.

According to Schaefer, et al. – Vol. I (2017):

> The generalized design process for embankment support is as follows:
>
> 1. Perform embankment design without stone columns to determine the overall settlement and global stability and to determine if stone columns or another form of ground modification are required. If yes proceed to Step 2.
>
> 2. Assume an area replacement ratio and column diameter.
>
> 3. Determine the spacing based on the assumed area replacement ratio and column diameter.
>
> 4. Check the load bearing capacity of the stone column to see if it meets the project requirements. If not revise the column diameter and re-check.
>
> 5. Determine the total settlement of the embankment supported on the stone columns.

6. Check the time rate of settlement. If the time for settlement is too large consider changing the column spacing.

7. Check global stability.

For the design procedure of Geopiers, the GEOR should review Schaefer, et al. Vol. I (2017). In addition, prior to the use of Geopiers, the GEOR shall obtain concurrence for both the use of Geopiers as well as the design methodology from the OES/GDS and the RPG/GDS.

19.4.9.1 Unit Cell Concept

According to Schaefer, et al. – Vol. I (2017)):

> 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 *19-9* and *19-10*. 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 *an effective* diameter (D_e) enclosing the tributary soil and *1* stone column is known as the "unit cell". The stone column is concentric to the exterior boundary of the unit cell.

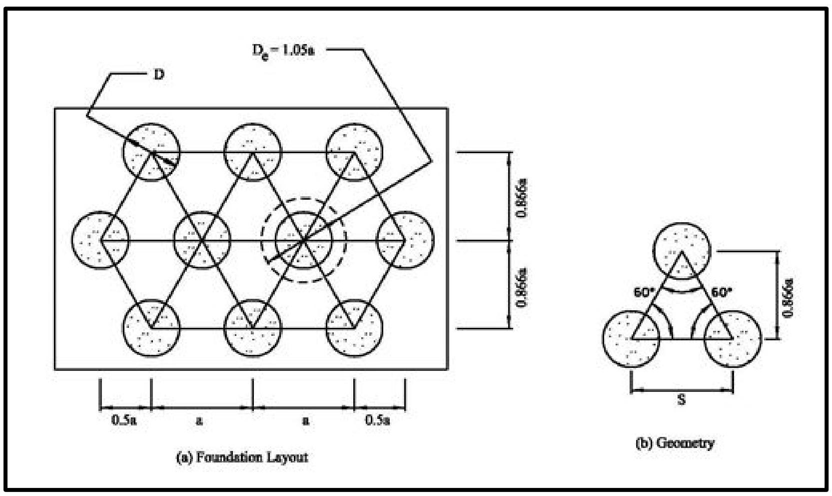


Figure 19-9, Stone Column Equilateral Triangular Pattern (Schaefer, et al. – Vol. I (2017))

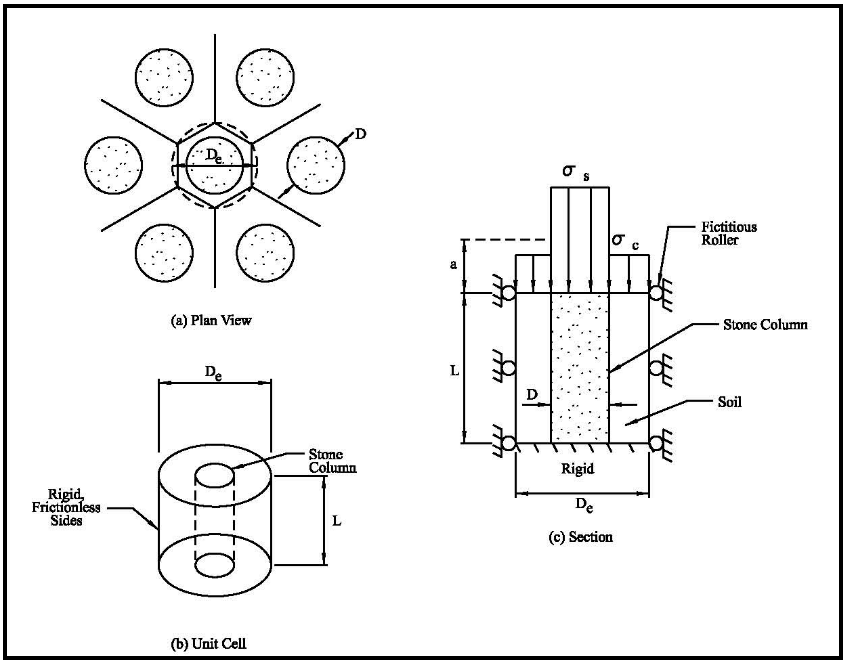


Figure 19-10, Unit Cell Idealization
(Schaefer, et al. – Vol. I (2017))

19.4.9.2 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.

$$\alpha_s = \frac{A_s}{A} = \frac{D^2}{D_e^2} \quad \textbf{Equation 19-7}$$

$$a_{ir} = \frac{1}{\alpha_s} = \frac{A}{A_s} = \frac{D_e^2}{D^2} \quad \textbf{Equation 19-8}$$

Where,

α_s = Area replacement ratio
A_s = Area of the stone column
A = Total area within the unit cell
a_{ir} = Area improvement ratio
D = Diameter of stone column (see Figure 19-10)
D_e = Effective diameter of unit cell (see Figure 19-10)

19.4.9.3 Spacing and Diameter

According to Schaefer, et al. – Vol. I (2017)):

> 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 12.0 feet. For footing support, *the stone columns* are installed in rows or clusters. For *either* footing or wide area support, *the stone columns* may extend beyond the loaded area.

19.4.9.4 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:

$$n = \frac{\sigma_{sc}}{\sigma_c} \qquad \textbf{Equation 19-9}$$

Where,

n = Stress ratio or stress concentration
σ_{sc} = 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 (3.0 to 4.0) may be required in very weak soils and when the column spacing is tight. Lower values of n (2.0 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.

$$q = (\sigma_{sc} * \alpha_s) + \sigma_c * (1 - \alpha_s) \qquad \textbf{Equation 19-10}$$

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.

$$\sigma_c = \frac{q}{[1+\alpha_s*(n-1)]} \qquad \textbf{Equation 19-11}$$

$$\sigma_{sc} = \frac{(n*q)}{[1+\alpha_s*(n-1)]} \qquad \textbf{Equation 19-12}$$

19.4.9.5 Additional Design Considerations

The procedures indicated in the previous Sections concern the design of stone columns as affected by the diameter, spacing, and distribution of stresses between stone columns and the surrounding soil. See Schaefer, et al. (2017) for the vertical capacity of stone columns, settlement, rate of settlement, shear strength increase caused by the installation of stone columns, and affect the installation of stone columns have on the seismic response of a site.

19.4.10 Geopiers®

Geopiers® shall be designed in accordance with the procedures detailed in Schafer, et al. (2017). In addition, prior to the use of Geopiers® on a SCDOT project, the acceptance of both the OES/GDS and the RPG/GDS is required.

19.4.11 Design Verification

According to Schaefer, et al. – Vol. I (2017)):

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

19.5 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 40 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 19-11 provides a schematic of dynamic compaction.

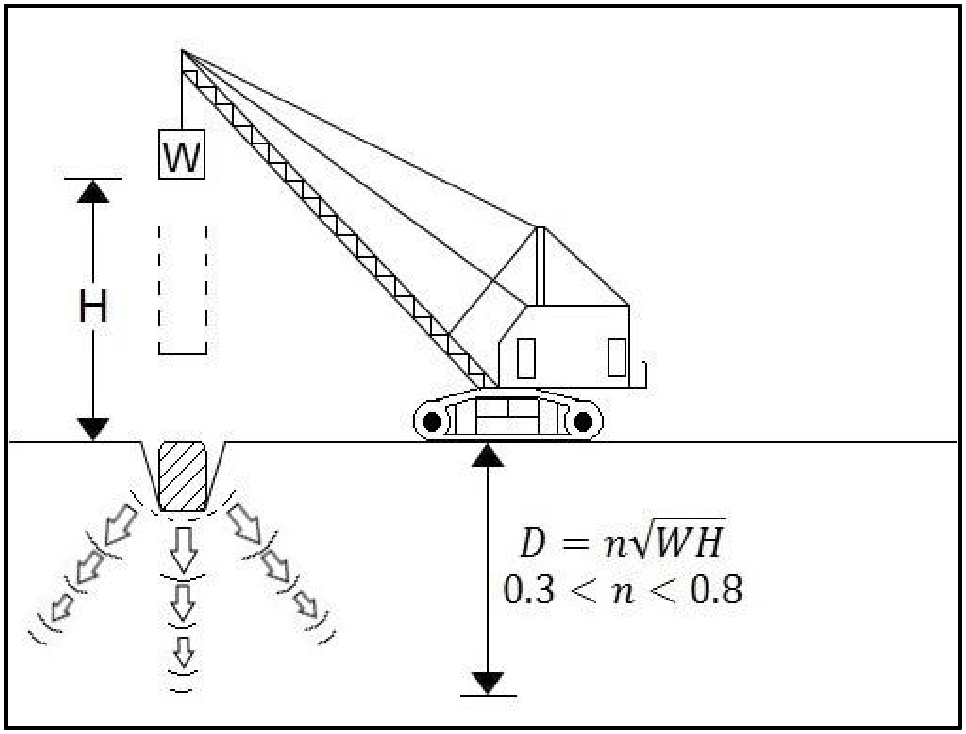


Figure 19-11, Dynamic Compaction Schematic
(Schaefer, et al. – Vol. I (2017))

19.5.1 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, Sand-Like soils (Zone 1 - sands, gravels, and non-plastic silts) that meet the gradation, permeability (hydraulic conductivity), and plasticity shown in Figure 19-12. For saturated Zone 1 soils, the induced excess pore pressures from dynamic compaction can 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.

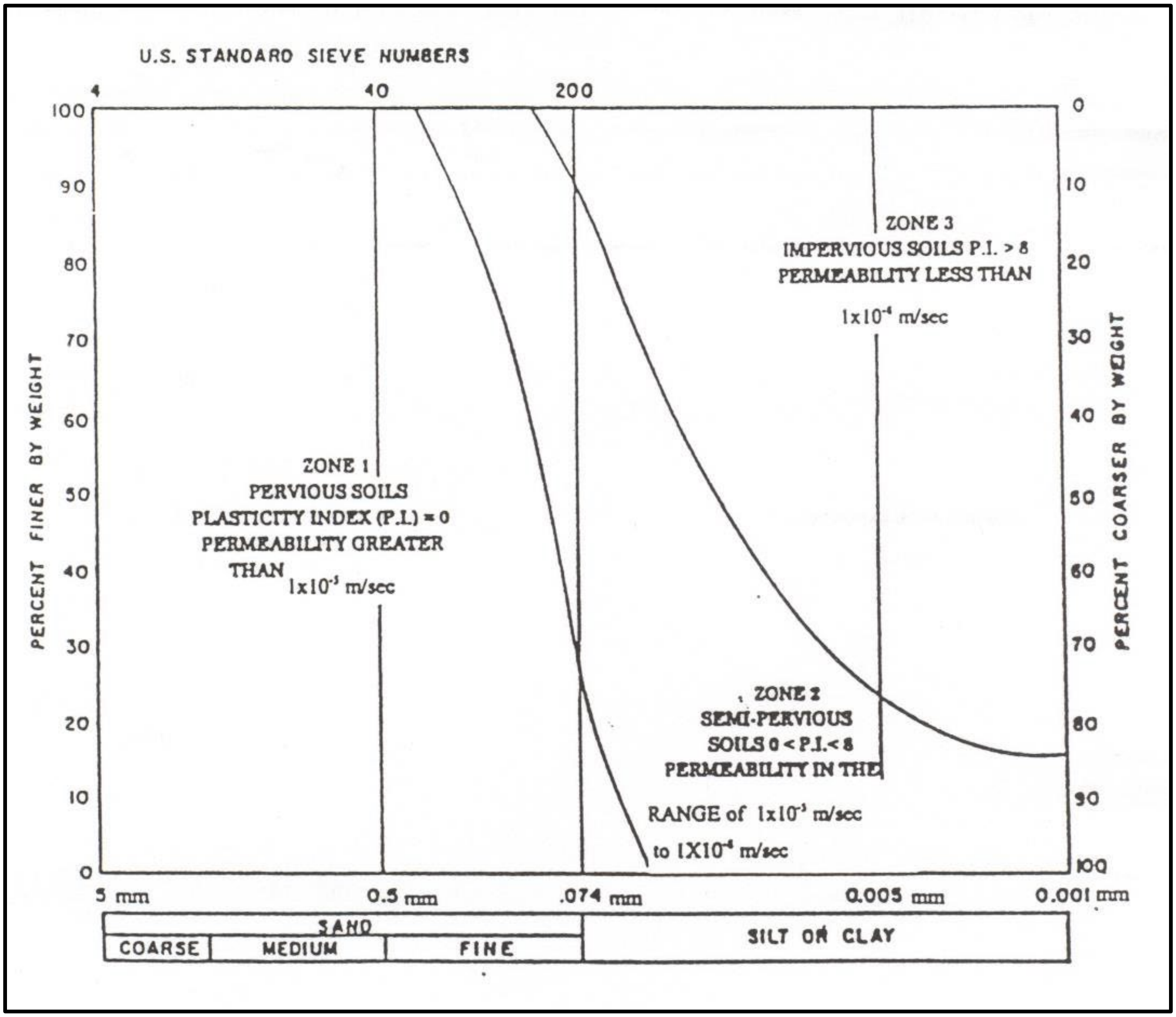


Figure 19-12, Soil Grouping for Dynamic Compaction (Schaefer, et al. – Vol. I (2017))

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 19-13). It should be noted that while the void ratio decreases, the volume of the solids does not change.

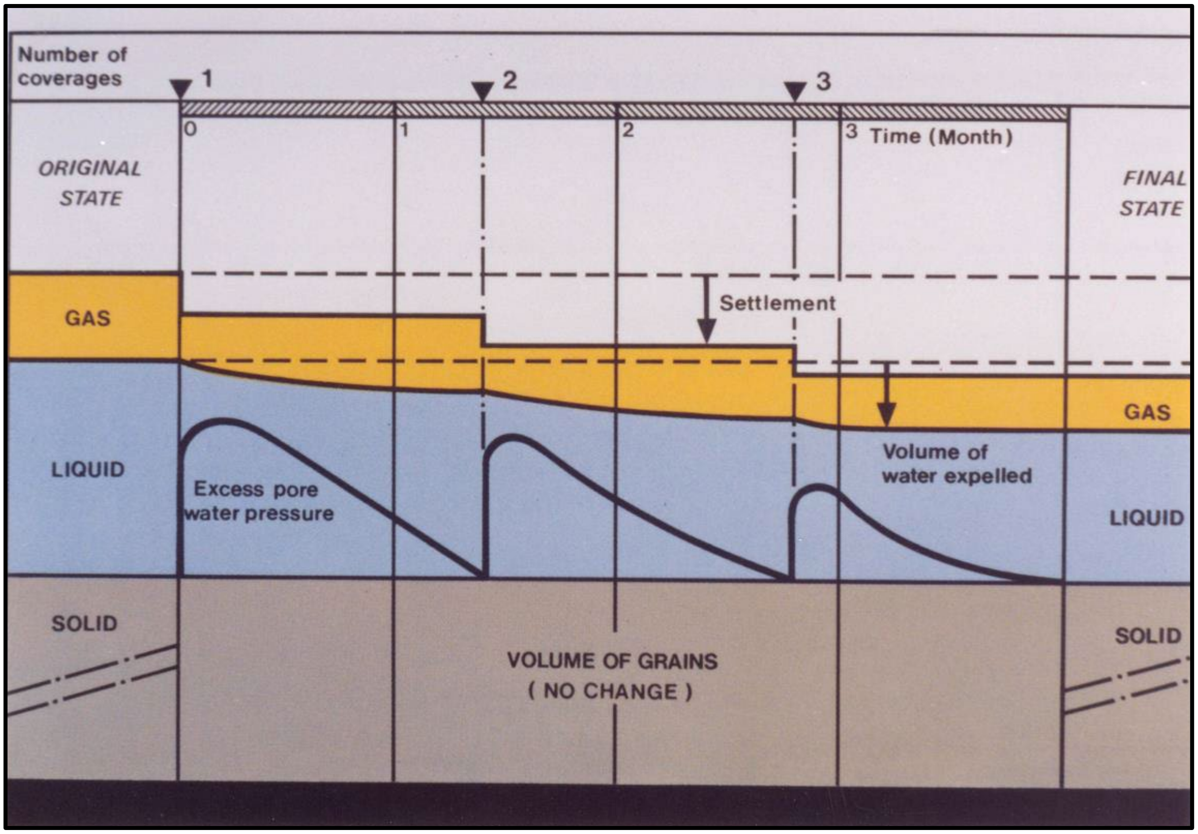


Figure 19-13, Dynamic Compaction Phase Diagram (Schaefer, et al. – Vol. I (2017))

The soils indicated in Zone 3 (Figure 19-12) are typically impervious, Clay-Like 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. Further, 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 Schaefer, et al. – Vol. I (2017)):

> 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. 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.

19.5.1.1 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.

19.5.1.2 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 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.

19.5.1.3 Environmental Considerations

As indicated previously the vibrations created by dynamic compaction can have an adverse effect on adjoining properties. Therefore determine the potential impact of vibrations caused by Dynamic Compaction using the procedures provided in Chapter 24.

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.

19.5.2 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.

If additional design guidance or information, is needed see Lukas (1995).

19.5.2.1 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 dynamic compaction using either SPT or CPTu 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.

19.5.2.2 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.

$$\boldsymbol{D_{max} = n\sqrt{W * H}}$$

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.

19.5.2.3 Degree of Improvement

As indicated above, the degree of improvement is typically measured using either SPT or CPTu measurements, which are performed prior to and after dynamic compaction to monitor the amount of improvement imparted on the soil. The confirmation testing should be performed after the dissipation of pore pressures induced by dynamic compaction. Figure 19-16 provides a general indication of the amount of improvement from dynamic compaction.

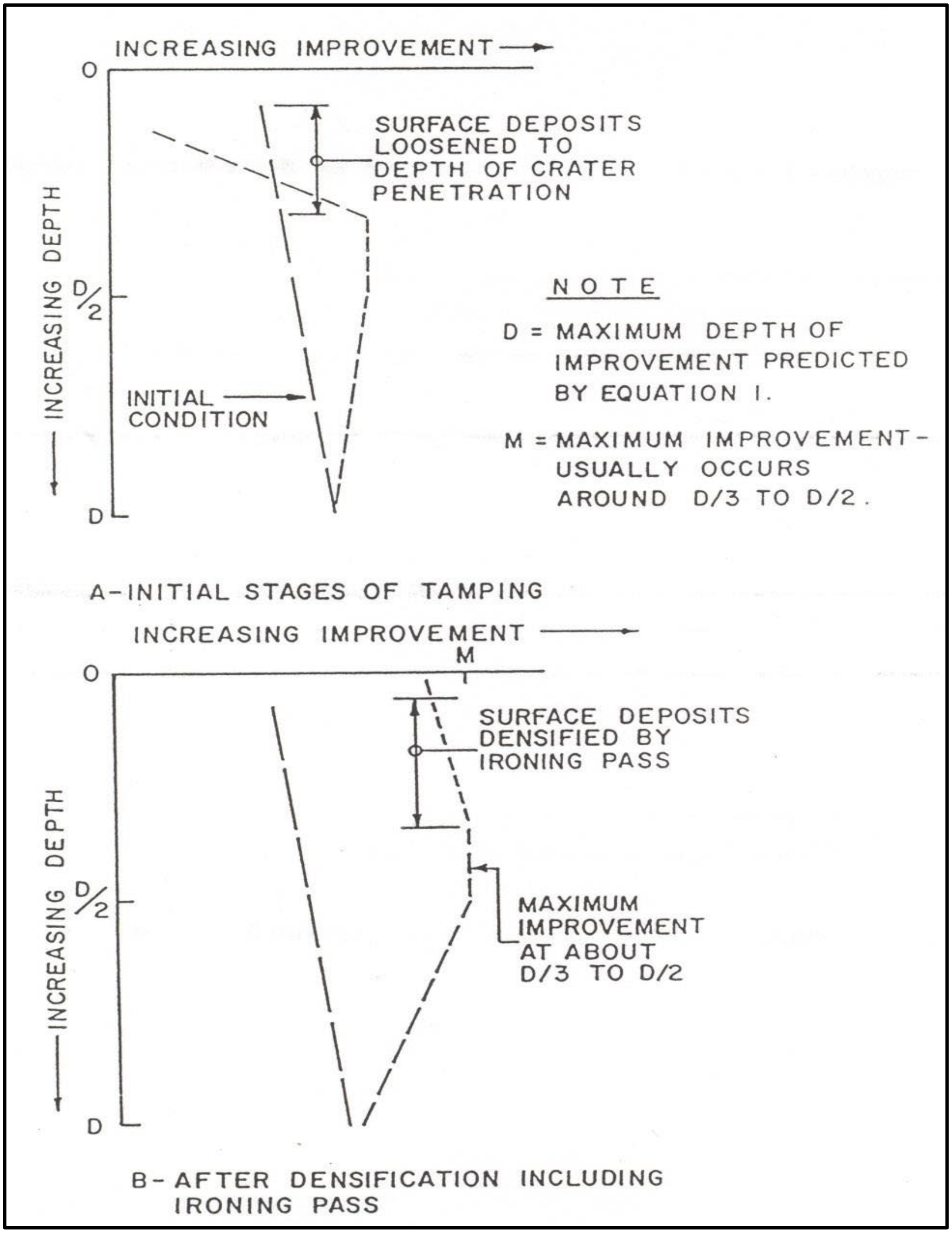


Figure 19-14, Dynamic Compaction Improvements vs. Depth (Schaefer, et al. – Vol. I (2017))

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 CPTu values that can be achieved. These maximum values are listed in Table 19-14. 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 19-14, Upper Bound Test Values after Dynamic Compaction (Schaefer, et al. – Vol. I (2017))

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.

19.5.2.4 Energy Requirements

According to Schaefer, et al. – Vol. I (2017), "Deep 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:"

$$AE = \frac{W*H*N*P}{G^2} \qquad \textbf{Equation 19-14}$$

Where,

AE = Applied energy
W = Tamper weight
H = Drop height
N = Number of drops at each specific drop point location
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 19-15.

Table 19-15, Applied Energy Guidelines
(Elias, et al. – Vol. I (2006))

Soil Deposit	Unit Applied Energy (ft-lb/ft^2)	Percent Standard Proctor Energy[1]
Zone 1 Soils[2]	4,100 – 5,200	33 - 41
Zones 2 and 3[2]	5,200 – 7,200	41 - 60
Landfills	12,400 – 22,700	100 - 180

[1]Standard Proctor energy equals 12,400 ft-lb/ft^2

[2]Refer to Figure19-12

19.6 DEEP MIXING METHODS

Deep mixing methods (DMM) are a ground improvement technique that mixes binders (i.e., cement, gypsum, blast furnace slag, fly ash, lime, or other hardening reagents) into the soil at a specific depth to improve the in-situ soil properties without requiring excavation or removal. DMM mixes the soil and binder (reagent) together, whereas grouting injects cementitious materials into the in-situ soil matrix to improve the soil. Grouting is discussed in a subsequent Section. Mass mixing methods (MMM) are a subset of the DMM technology and can be used for a variety of applications including excavation support, soil stabilization, settlement reduction, foundation support, and mitigation of liquefaction potential. There are however differences between DMM and MMM, those differences are indicated below:

1. The percentage area coverage for MMM is 100 or nearly 100 percent.
2. The design strength of the MMM soil is less than the DMM soil.
3. The depth of treatment for MMM soil is limited to approximately 30 feet beneath the existing ground surface.

Because of the similarities between DMM and MMM, other than as indicated previously, DMM will be used generically for the remainder of this Section unless specifically indicated. Detailed design guidance for DMM is available from Bruce, et al. (2013). DMM 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 19-16, Deep Mixing Generic Terms
(Bruce, et al. (2013))

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

These generic terms can be combined into 5 distinct processes of deep soil mixing (see Figure 19-17), WRS, WRE, WJE, WVP, and DRE. Some of the possible combinations of deep soil mixing methods do not exist. For example DJE (dry, jet, end) does not exist.

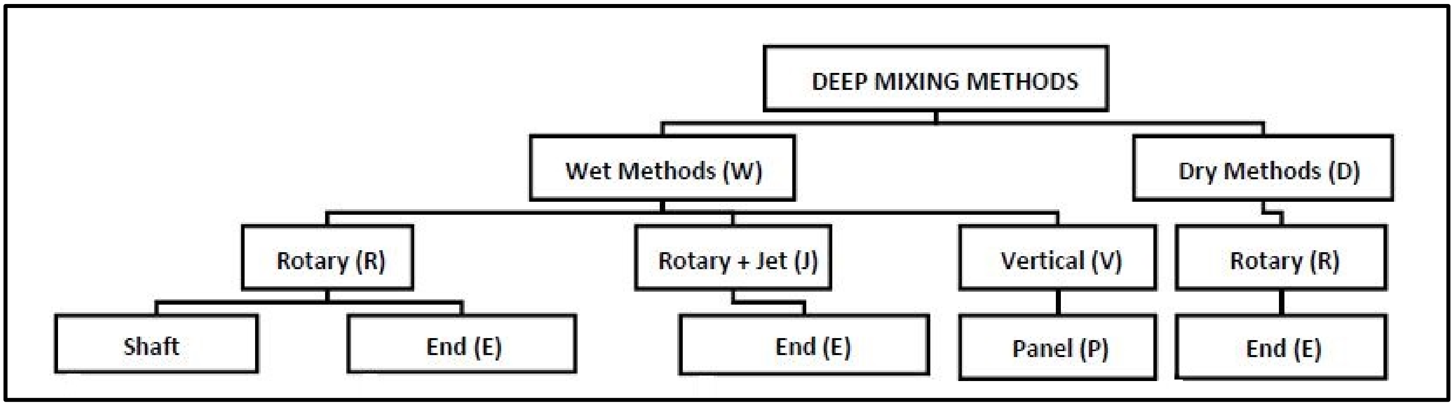


Figure 19-15, Generic Classification of Deep Mixing Techniques (modified from Bruce, et al. (2013))

The 5 processes discussed previously can be divided into 2 groups as indicated in Table 19-17.

Table 19-17, DMM Groups

Wet DMM	WRS – Wet Rotary Shaft	Single or multiple shaft equipment with blades over a length of the shaft that mechanically mix injected slurry with surrounding soil.
	WRE – Wet Rotary End	Single shaft equipment with singe mixing tool.
	WJE – Wet Jet End	Single (uncommon) or multiple shaft equipment tipped with blades and assisted by jetting of slurry through high-pressure ports
	WVP – Wet Vertical Panel	Chainsaw-type vertical cutting tool mounted on a central cutter post.
Dry DMM	DRE – Dry Rotary End	Single-auger column technique developed for soil stabilization and reinforcement of cohesive soils. Binder is inserted into the soil via compressed air (jet).

DMM can be performed wet or dry and is generally done using large-diameter, single-axis, vertical-shaft mixing equipment for the wet method. In the dry method the binder is delivered to the mixing/cutting head via compressed air. Dry DMM is typically performed in soft, saturated, or nearly saturated soil. Wet DMM can be applied to soils with any degree of saturation.

19.6.1 Analysis

Regardless of whether wet or dry DMM or MMM is used, all DMM projects should follow the flowchart provided in Figure 19-18. Wet construction DMMs are typically used for large-scale structural support improvement using individual elements, shear walls, or grid type arrangements (see Figures 19-19 and 19-20), while dry DMMs are used primarily for soil stabilization/reinforcement and settlement reduction (i.e., MMM). While DMM provide vertical (compressive) capacity, reduce settlement, increase stiffness, there is limited to no tensile resistance from these materials. Therefore, there is no tensile resistance allowed for DMM by itself. Like any other soil material DMM will provide axial resistance to other structural elements. DMM and MMM can be combined to create load-transfer type platform similar to those used for column supported embankments (see Figure 19-21). Discussed in the following paragraphs are applications, of wet and dry deep soil mixing that are typical for transportation related projects.

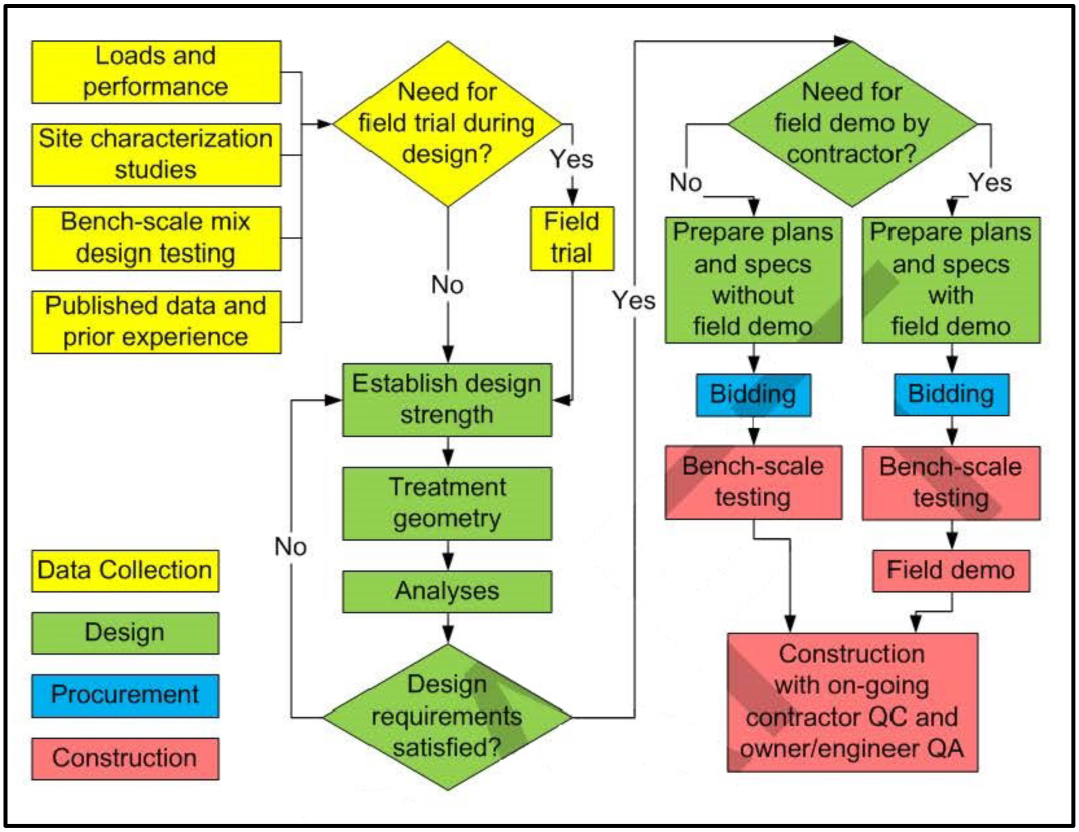


Figure 19-16, DMM Project Flowchart
(Schaefer, et al. – Vol. II (2017))

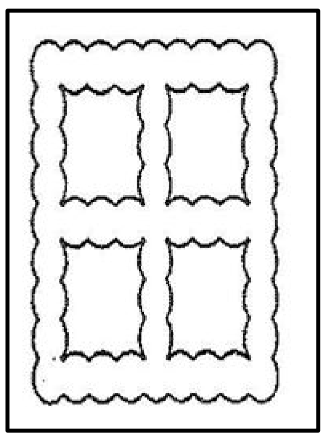


Figure 19-17, DMM Grid Treatment Pattern
(Elias, et al. – Vol. I (2006))

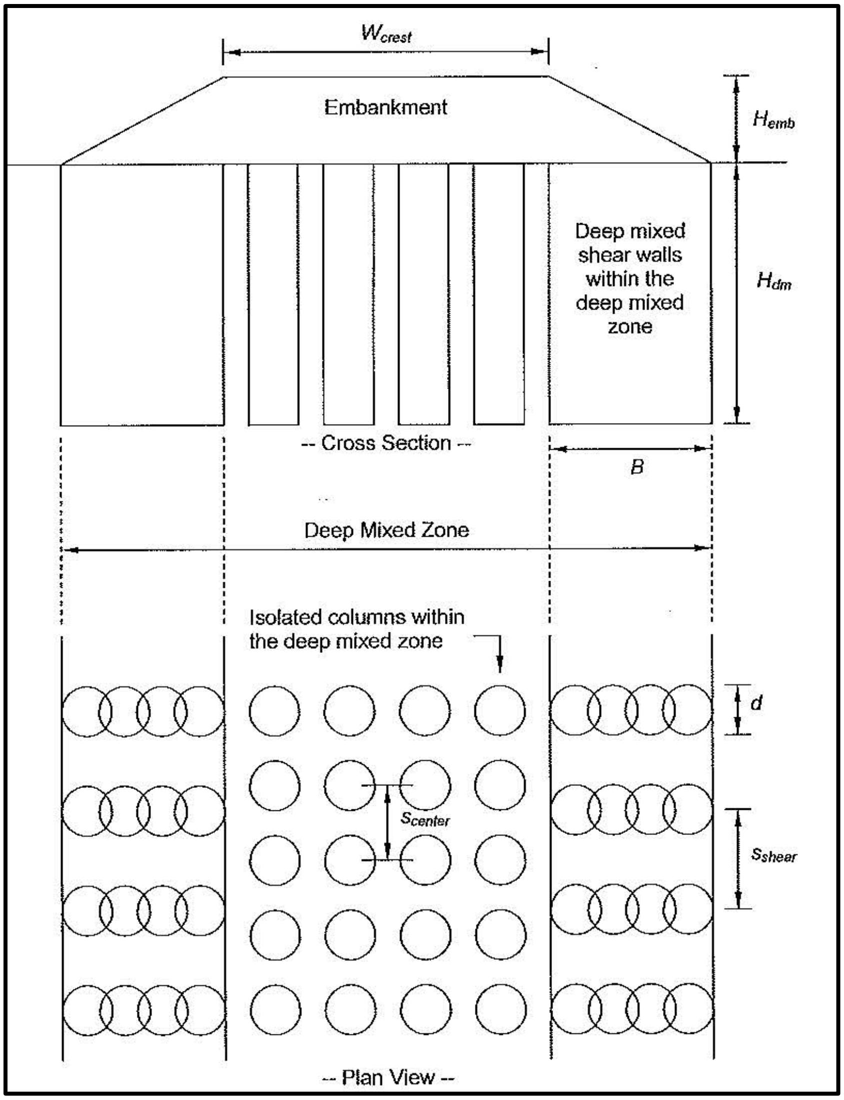


Figure 19-18, DMM Treatment Pattern Beneath Embankment (Bruce, et al. (2013))

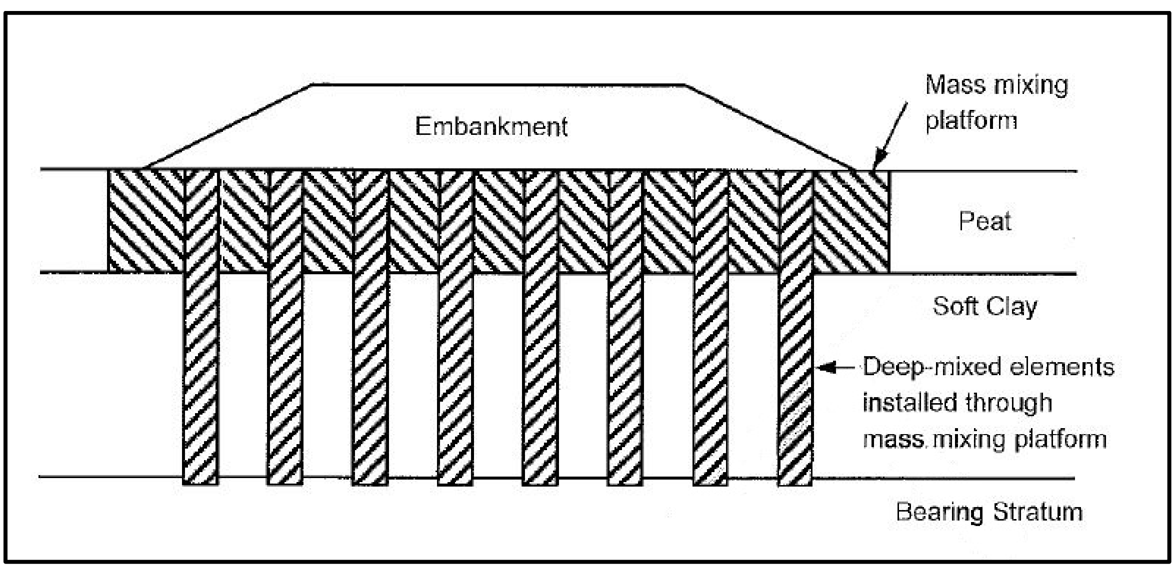


Figure 19-19, DMM-MMM Load-Transfer Platform (Schaefer, et al. – Vol. II (2017))

Wet DMM can be single- or multi-shaft processes that use cement-based slurries to create isolated elements, continuous walls, or blocks. Dry DMM typically use single-auger techniques that use lime, cement, lime-cement, or slag mixtures to create isolated columns, walls, or blocks for soil stabilization and reinforcement.

19.6.2 DMM Advantages and Disadvantages/Limitations

19.6.2.1 DMM Advantages

According to Schaefer, et al. – Vol II (2017) DMM has the following advantages:

- Increases the strength and decreases the compressibility of soft silts, clays and organic soils, and peat.
- Prevents liquefaction of loose sand deposits.
- Wet-mixing equipment can penetrate layers of dense and strong material to treat underlying weak, loose, or compressible layers.
- Improves soft clay deposits more quickly than using *PVDs* with preloads and surcharge.
- Permits reduced embankment footprint and fill volume through use of steeper side slopes or vertical walls.
- The plan view arrangement of treatment, the treatment depth, and the degree of improvement to strength and stiffness can be easily adjusted to satisfy design requirements and subsurface conditions.
- Carries new loads placed adjacent to existing facilities so the new loads do not cause settlement of the existing facilities.
- High production capacity with large equipment.
- Materials are treated in-situ, which can reduce disposal problems:
    - The dry method produces very little to no spoils.
    - Spoils from the wet *DMM* make excellent fill material.
- Stabilizes many types of contaminants *(typically not a reason why DMMs are constructed for SCDOT projects)*.
- Can be used for dry land and marine projects.
- Economical on large projects.
- Dewatering is not necessary.
- Less noise and vibrations than from some other technologies.
- *Specific advantages to MMM:*
    - *MMM is typically less expensive than traditional DMM techniques* on a unit volume basis, although the treatment per foot of depth is larger because of the larger area replacement ratio.
    - *MMM* can be done rapidly.

19.6.2.2 DMM Disadvantages/Limitations

According to Schaefer, et al. – Vol II (2017) DMM has the following disadvantages/limitations:

- The mobilization and unit costs can be higher than for other technologies, such as *PVDs* with preloading.

- *DMM* requires familiarity of the *GEOR* with specialized design, construction, specifications, and QC/QA practices.
- Cobbles, boulders, dense sand deposits, buried logs, and other obstructions can interfere with penetration of mixing equipment.
- Buried utilities and structures must be avoided. If buried features cannot be spanned and if treatment immediately adjacent to them is necessary, another technology, such as jet grouting, may be required.
- The wet *DMM* generally uses heavy equipment, which can require timber mats or other techniques to enable equipment to operate on soft ground.
- For the wet *DMM*, if there is not an opportunity for on-site use of the good quality fills generated by the spoils, the spoils may have to be transported off site for use on another project or to be disposed.
- *DMM* elements can only be installed vertically.
- Treatment depths are typically limited to about 130 feet.
- *Specific disadvantages/limitations to MMM:*
    - Treatment depths are typically limited to about 50 feet for shallow soil mixing equipment and to about 30 feet for *MMM* equipment *(i.e., mixing drum attached to backhoe stick)*.
    - *MMM* equipment typically cannot penetrate dense or stiff soils, cobbles, boulders, obstructions, and buried utilities or structures.
    - Quality control operations for MMM are not usually as sophisticated as for modern *DMM*. The quality and uniformity of the finished product is more operator dependent for *MMM* than for *DMM*.

19.6.3 <u>Feasibility</u>

The feasibility of using DMM 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 a bench-scale treatability study. Typically DMM is performed on very soft to firm clays, very loose to medium dense sands, very soft to firm organic soils and peats. Wet DMM equipment tends to more powerful than dry DMM equipment; therefore, wet DMM equipment is more likely to penetrate layers of stiff clay and dense sands to reach underlying soft/loose soil layers or organics. DMM may be used to treat contaminated soils in-situ by immobilizing the contaminants. Wet DMM requires more space than does dry DMM for an equipment yard, slurry batch plant, and equipment maneuvering. If the near surface soils are soft then a working platform may be required to support not only the DMM equipment, but also the slurry batch plant for wet DMM as well as storage for the binder.

19.6.3.1 Site Investigation

The site investigation required for DMM exceeds the requirements previously indicated in this Manual. If deep soil mixing is selected or proposed as an alternate ground improvement method, then, additional site-specific information will be required. The proposed site investigation plan shall be submitted to the RPG/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 DMM 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. Further, the proposed site investigation plan shall include the methodology for obtaining the required amount of material for bench-scale treatability study. 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
- Organic content and loss on ignition
- 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

19.6.3.2 Feasibility Assessment

DMM is best used when the subsurface conditions are soft to loose, with no obstructions, to depths no greater than 130 feet. There should be unrestricted overhead clearance and a need for relatively vibration free ground improvement methods. DMM 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 19-18, Favorable Soil-Chemistry Factors (Bruce, et al. (2013))

Property	Favorable Soil Chemistry
Near surface temperature	$\geq$ 39° F
pH	$>$ 5
Natural moisture content	$<$ 200 % (dry DMM) $<$ 60 % (wet DMM)
Organic content	$<$ 6 % (wet DMM)
Loss on Ignition	$<$ 10 %
Humus Content	$<$ 1 %
Electrical conductivity	$\geq$ 1.2 mΩ/cm

19.6.3.3 Bench-Scale Treatability Study

After assessing the viability of soil for DMM, samples should be prepared to determine the water, soil, binder (reagent) ratios as well as determining the time required for mixing. A bench-scale treatability study shall be performed during the additional exploration phase. Enough of the targeted material for DMM should be obtained, ranging from a minimum of 35 to more than 70 pounds. A minimum of 5 sets of 8 2- by 4-inch cubes shall be required to determine shear strength for each mix design proposed. The samples should then be tested for unconfined compressive strength at various curing times to determine strength gains with time. The bench-scale treatability study 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. A more detailed discussion of the bench-scale treatability study is provided in Bruce, et al. (2013).

Table 19-19, Typical DMM Improved Engineering Properties (Bruce, et al. (2013))

Property	Typical Range
Unconfined Compressive Strength, q_u	Dry DMM – 2 – 400 psi Wet DMM – 20 – 4,000 psi
Hydraulic Conductivity, k	Wet DMM – 10^{-5} – 10^{-6} cm/s
Young's Modulus (E_{50}) [Secant Modulus at 50% q_u]	Dry DMM – 150 q_u Wet DMM – 300 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 19-20, Mixing Guidelines (Elias, et al. – Vol. I (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

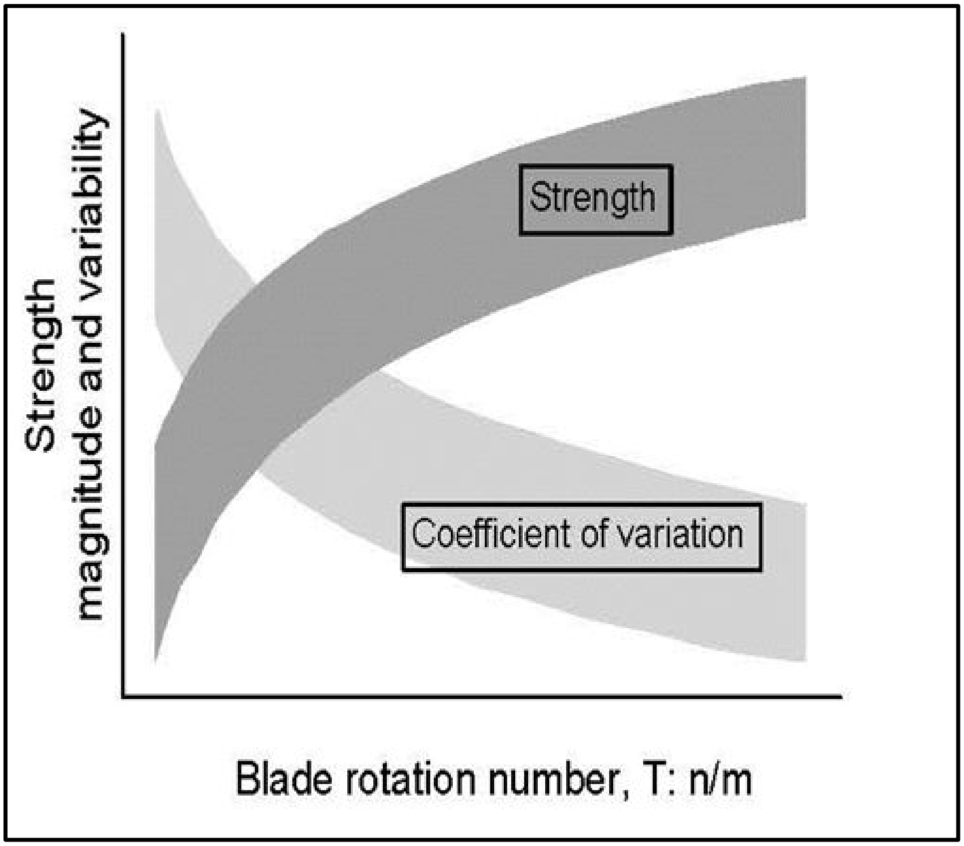
Bruce, et al., (2013)) have developed an "simplified index" factor, BRN (Blade Rotation Number), that quantifies the number of mixing cycles per meter which relates the penetration and retrieval speed (velocity) and the rotation speed during penetration and retrieval. BRN is defined as the total number of rotations during 1 meter of penetration (downstroke) or withdrawal (upstroke) after the binder (reagent) has been added into the ground. Use Equation 19-14 to determine the BRN.

$$\boldsymbol{BRN} = \boldsymbol{\Sigma \mathrm{M}} * \left[\frac{\boldsymbol{N_p}}{\boldsymbol{V_p}} + \frac{\boldsymbol{N_w}}{\boldsymbol{V_w}}\right] \qquad \textbf{Equation 19-15}$$

Where,

ΣM = Total number of mixing blades
V_p, V_w = Penetration and Withdrawal mixing blade velocity (meters/min)
N_p, N_w = Blade rotation speed during penetration and withdrawal (rpm)
1 meter = 3.2808 ft

BRN greater than 360 for wet DMM tend to have a smaller coefficient of variation. In addition Larsson, (2005), indicates that "…the retrieval rate and the number of blades have a significant influence on the strength magnitude and variation." An approximate logarithmic relationship appears to exist between the strength of the DMM and the mixing energy (see Figure 19-22). However, the strength magnitude cannot be based solely on the mixing energy, but also depends on the reagent, soil, and in-situ condition during curing.



BRN = T

Figure 19-20, T vs Strength Magnitude and Variability (Larsson (2005))

19.6.4 Design

DMM shall be designed following the procedures indicated in Bruce, et al. (2013). The GEOR shall determine the required DMM strength based on the needs of the project and the bench-scale treatability study. Strength as used here typically refers to shear strength, while during QC/QA strength typically refers to unconfined compressive strength. The GEOR shall indicate on the plans which strength is being required for the project. The geometric parameters listed in Table 19-21 shall be determined by the GEOR and provided on the DMM plan sheet.

Table 19-21, DMM Required Geometric Parameters (modified Bruce, et al. – Vol. I (2013))

Common to both Isolated Columns and Shear Walls		
Parameter		**Maximum and/or Minimum**
Top elevation of DMM element		Minimum
Bottom elevation of DMM element		Maximum
d	Column diameter or shear wall thickness (see Figure 19-19 and Figure 19-23)	Minimum and maximum
Isolated Columns		
Parameter		**Maximum and/or Minimum**
s_{center}	Center-to-center spacing of isolated columns (see Figure 19-19)	Maximum
$s_{center} - d$	Edge-to-edge spacing of isolated columns	Maximum
$\alpha_{s,\ center}$	Area replacement ratio beneath central portion of embankment (see Figure 19-19)	Minimum
Shear Walls		
Parameter		**Maximum and/or Minimum**
B	Length of shear wall (see Figures 19-19 and 19-23)	Minimum
b	Average shear wall width (see Figure 19-23)	Minimum
e	Overlap distance (see figure 19-23)	Maximum
e/d	Ratio of overlap distance to column diameter	Minimum
s_{shear}	Center-to-center spacing of shear walls (see Figures 19-19 and 19-23)	Maximum
$s_{shear} - d$	Edge-to-edge spacing of shear walls	Maximum
$\alpha_{s,shear}$	Area replacement ratio beneath side slopes embankment (see Figure 19-19)	Minimum
c	Chord length (see Figure 19-23)	Maximum
c/s_{shear}	Ratio of chord length to Edge-to-edge spacing of shear walls	Minimum

The area replacement ratio central portion of an embankment (see Figure 19-19) or for isolated columns is determined using the following equation,

$$\alpha_{s,center} = \frac{\pi * d^2}{4 * (s_{center})^2}$$ **Equation 19-16**

The area replacement ratio for shear walls beneath an embankment (see Figure 19-23) is determined using the following equation,

$$\alpha_{s,shear} = \frac{b}{s_{shear}}$$ **Equation 19-17**

The area replacement ratio for overlapping columns is influenced by the extent of the overlap between the columns and is determined using the following equation,

$$\alpha_{s,shear} = \frac{\pi * d * (1 - a_e)}{4 * s_{shear} * (1 - \frac{e}{d})}$$ **Equation 19-18**

$$a_e = \frac{\beta - \sin\beta}{\pi}$$ **Equation 19-19**

$$\beta = 2 * arcsin\,(1 - \frac{e}{d})$$ **Equation 19-20**

Note that a_e is the overlap area ratio and β is the chord angle expressed in radians (1° = (π/180)). The chord length, c, is determined using the following equation,

$$c = d * \ sin\left(\frac{\beta}{2}\right)$$ **Equation 19-21**

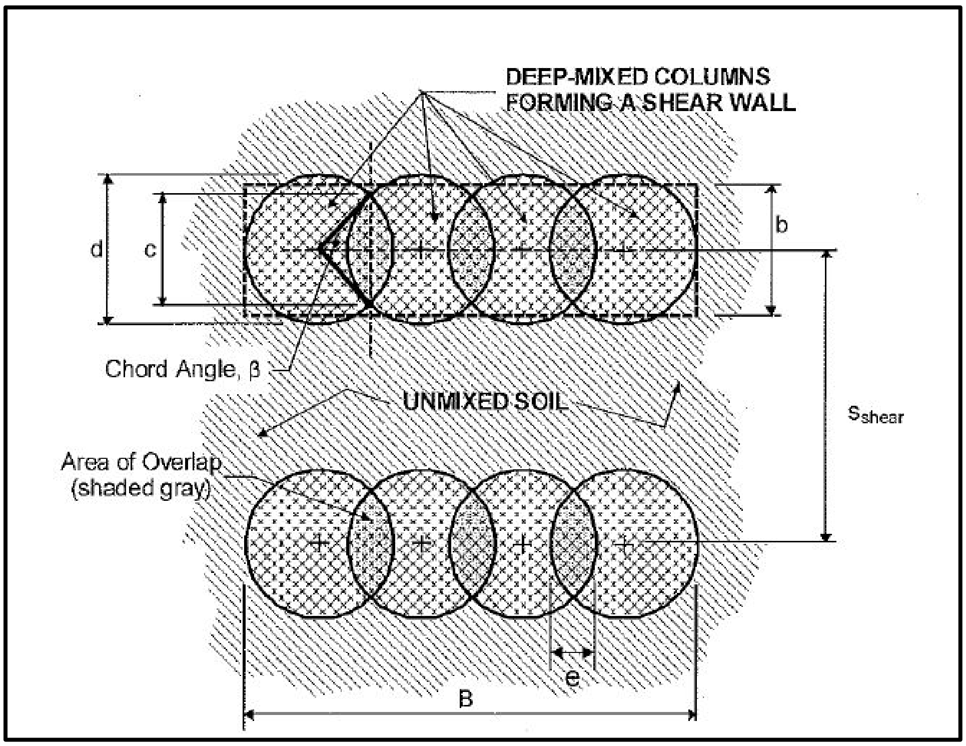


Figure 19-21, DMM Shear Wall Geometric Detail (Bruce, et al. (2013))

19.6.5 Verification

The properties of the improved ground require verification to ascertain whether the requirements of the project are being met. The contractor shall 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 electro-piezocone penetrometer testing (CPTu), dilatometer testing (DMT), standard penetration testing (SPT), or pressuremeter testing (PMT).

19.6.6 Construction Considerations

The GEOR is required to prepare a Special Provision for Deep Mixing Methods. The GEOR shall assure that the Special Provision contains definitions for terms used in both the Special Provision as well as the drawings. In addition, to preparing the Special Provision, the GEOR shall prepare all construction drawings.

19.7 GROUTING

According to Schaefer, et al. – Vol. II (2017);

> Grouting comprises a variety of techniques that employ injection of a range of materials into soil or rock formations, via boreholes, to improve their engineering properties. 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 Schaefer, et al. – Vol. II (2017) are used in this Manual. Schaefer, et al. – Vol. II (2017) identifies 2 principle types of grouting which are listed in the table below. Figure 19-24 provides schematics of the various types of grouting.

Table 19-22, Types of Grouting Method
(Schaefer, et al. – Vol. II (2017))

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)
	Low mobility grouting - Compaction or displacement and bulk void filling
	Jet (or replacement)
	Fracture (including compensation grouting)
	Slabjacking

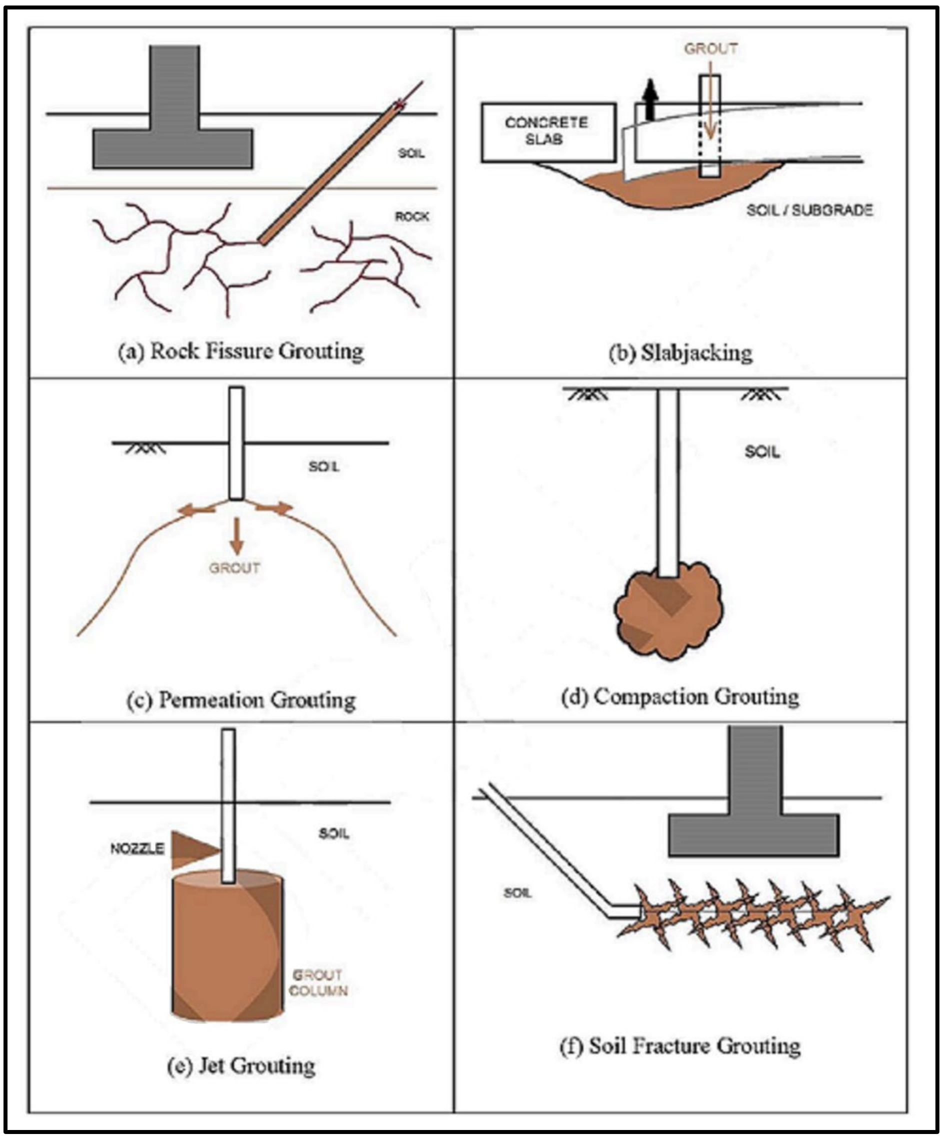


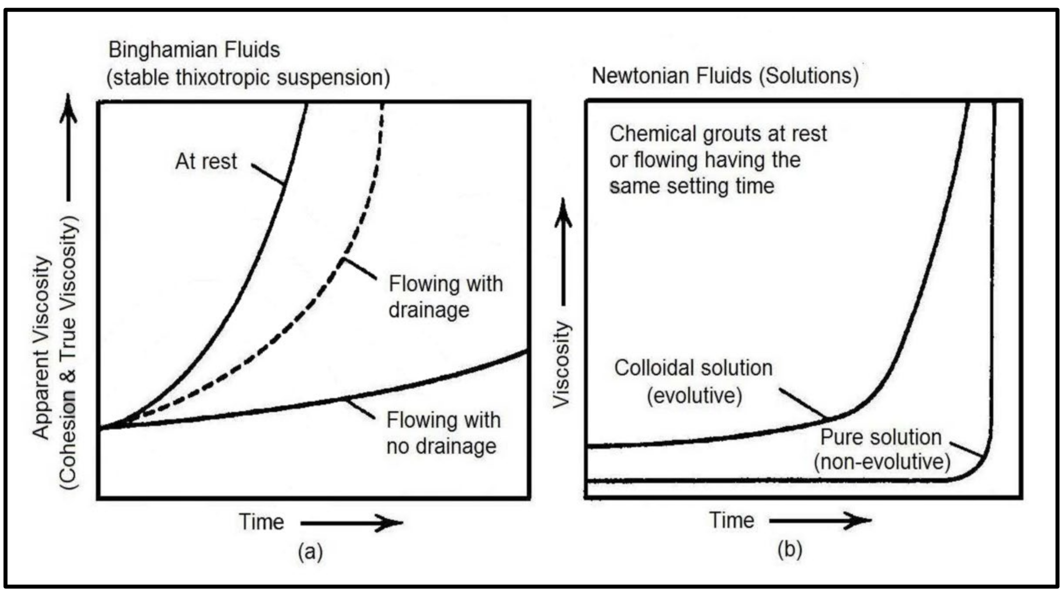
Figure 19-22, Types of Grouting Schematic
(Schaefer, et al. – Vol. II (2017))

19.7.1 Grout Materials

There are 4 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 19-23(a) shows the increase in apparent viscosity with time for these grouts and Figure 19-24 shows grain-size distribution of various cements.



(a) Category 1 Grouts; (b) Categories 2 and 3 Grouts

Figure 19-23, Viscosity versus Time
(Schaefer, et al. – Vol. II (2017))

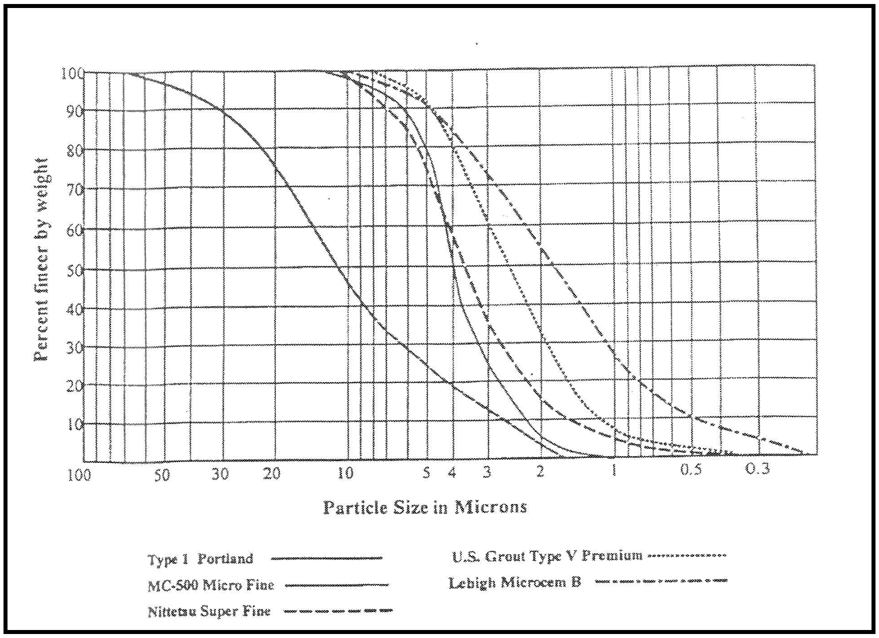


Figure 19-24, Grain-Size Distribution of Cements (Elias, et al. – Vol. II (2006))

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 19-23(b) 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, synæresis and durability, as well as cost and environment acceptability.

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 2 components (epoxies or aminoplasts)

The setting times for these grouts is adjusted by varying the proportions of the reagents or components. According to Schaefer, et al. – Vol. II (2017)):

> 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 4 subcategories as indicated in Table 19-23.

Table 19-23, Types, Use, and Applications of Resin Grouts (Schaefer, et al. – Vol. II (2017))

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 220 psi Strengthening of a granular medium subjected to vibrations
Phenol	Granular, very fine soils	Strengthening
Aminoplastic	Schists and coals	Strengthening (by adherence to materials of organic origin)
Polyurethane	Large Voids	Formation of a foam that forms a barrier against running water (using water-reactive resins) Stabilization or localized filling (using 2-component resins)

There are only 2 types of polyurethanes that are appropriate for grouting. These types are listed in Table 19-24.

Table 19-24, Polyurethane Types (modified Schaefer, et al. – Vol. II (2017))

Polyurethane Type	Properties
Water Reactive	Liquid resin reacts with groundwater to form either flexible (elastomeric) or rigid foam These resins take 2 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
2-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.

19.7.2 Rock Grouting

There are 2 types of rock grouting: rock fissure grouting and void filling. Both types of grouting are discussed briefly in the following sections.

19.7.2.1 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 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. 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 poor field practices 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.

$$\boldsymbol{N_R = \frac{f_w}{(D_{95})_{Grout}}} \qquad \textbf{Equation 19-22}$$

Where,

N_R = Groutability ratio of rock
f_w = Fissure width
$(D_{95})_{Grout}$ = 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 2. Rock grouting with particulate materials normally falls into 1 of the categories indicated in Table 19-25.

Table 19-25, Rock Grouting Categories
(Elias, et al. – Vol. II (2006))

Rock Grouting Category	Description
Curtain	Drilling and grouting of 2 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.

19.7.2.2 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. 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 OES/GDS for guidance in the use of this method for remediation of scour.

19.7.3 Soil Grouting

Soil grouting programs are used to achieve a variety of ground improvement objectives. The 2 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 5 different types of grouting that can be used on soil:

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

All 5 of these types of grouting can be used for water control, waterproofing, and structural enhancement and 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 19-25.

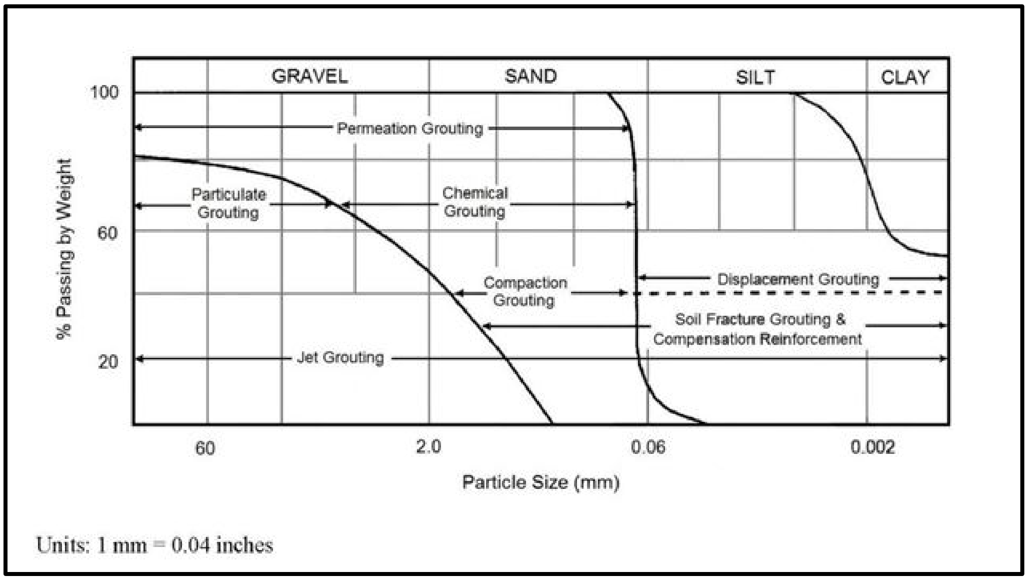


Figure 19-25, Range of Applicability of Soil Grouting Techniques (Schaefer, et al. – Vol. II (2017))

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 Elias, et al. – Vol. II (2006)):

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)

19.7.3.1 Site Investigation

The pregrouting subsurface exploration is more detailed than is normally required and should include continuous sample and laboratory tests. These tests should include 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.

19.7.3.2 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 19-26). 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 19-26 and Figure 19-27 provide the approximate percentage of material passing the #200 sieve and the groutability of a soil.

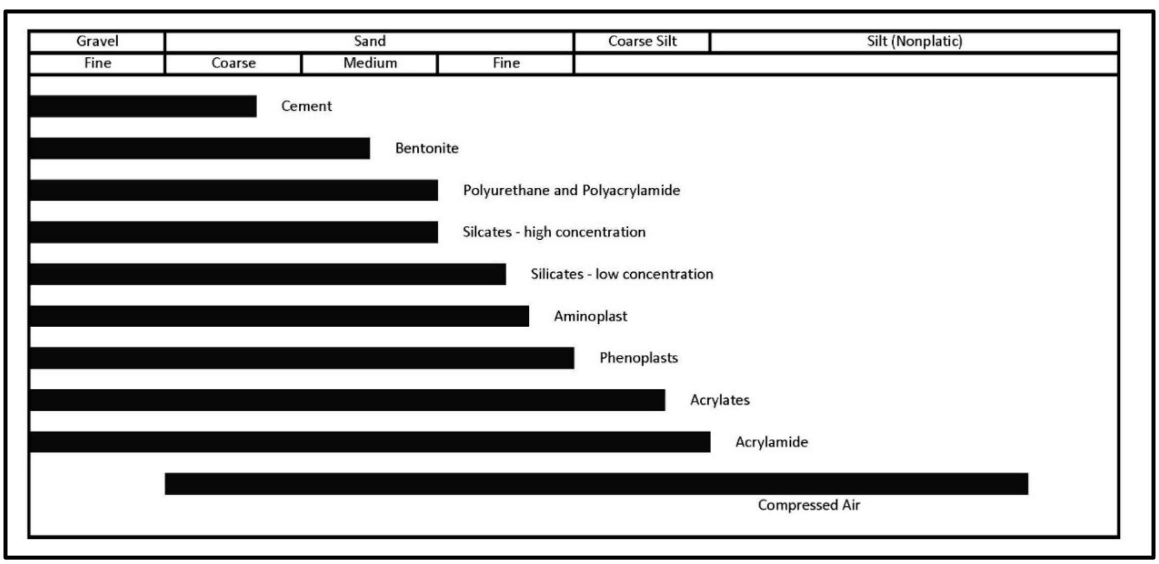


Figure 19-26, Penetrability of Various Grouts versus Soil Type (modified Elias, et al. – Vol. II (2006))

Table 19-26, Groutability Guidelines

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

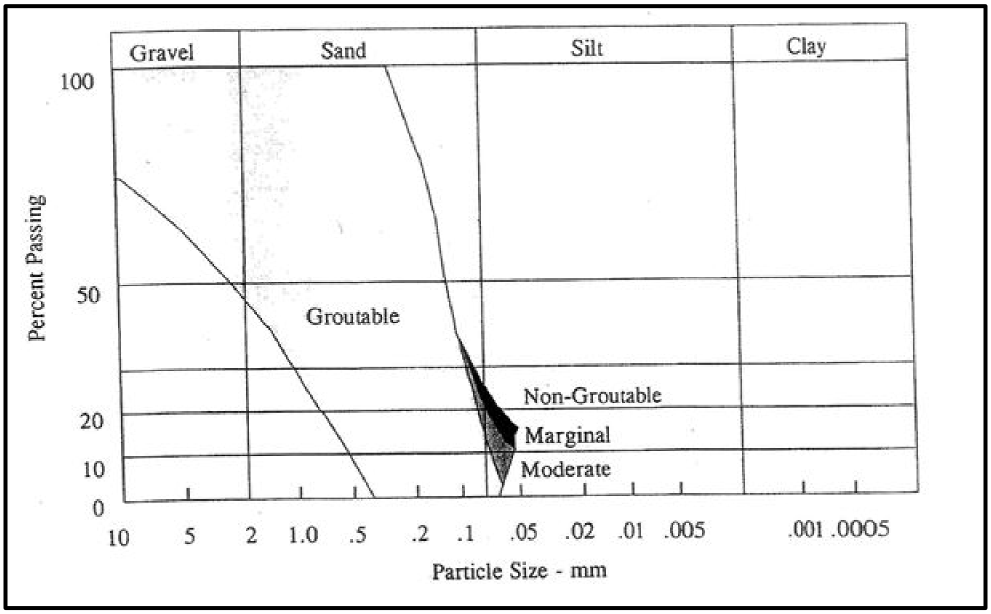


Figure 19-27, Grain-Size Distribution for Permeation Grouting (Elias, et al. – Vol. II (2006))

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.

$$\frac{(D_{15})_{soil}}{(D_{85})_{grout}} = \Psi \qquad \textbf{Equation 19-23}$$

$$\frac{(D_{10})_{soil}}{(D_{95})_{grout}} = \Theta \qquad \textbf{Equation 19-24}$$

Where,

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

Table 19-27, 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.

19.7.3.3 Compaction Grouting

According to Elias, et al. – Vol. II (2006):

> *Compaction grouting* consists of the injection of low slump *(usually 1 inch or less)*, low mobility grout into loose or loosened soils of appropriate grain-size distribution. …compaction grouting can be used in a wide variety of applications, including soil densification (for static and seismic enhancement), raising of surficial structures settlement control over…sinkholes and for structural underpinning. Compaction grout can also be used to seal off major water ingresses through open channel systems.
>
> *Figure 19-27* indicates the range of soils where densification by compaction grouting may be expected to be effective, i.e., in all relatively free-draining soils, including gravels, sands, and coarser silts. In fine-grained soils, pore pressures may not be able to dissipate and improvement may not be economically achievable. Grout mix design is also critical, in that the grout must have internal friction to ensure that the bulbs preserve their "spheroidal" shape in the soil. Otherwise, fracturing and lensing will occur, leading to ineffective densification.

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 3 to 12 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.

19.7.3.4 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 19-28 provides a schematic of the jet grouting procedure.

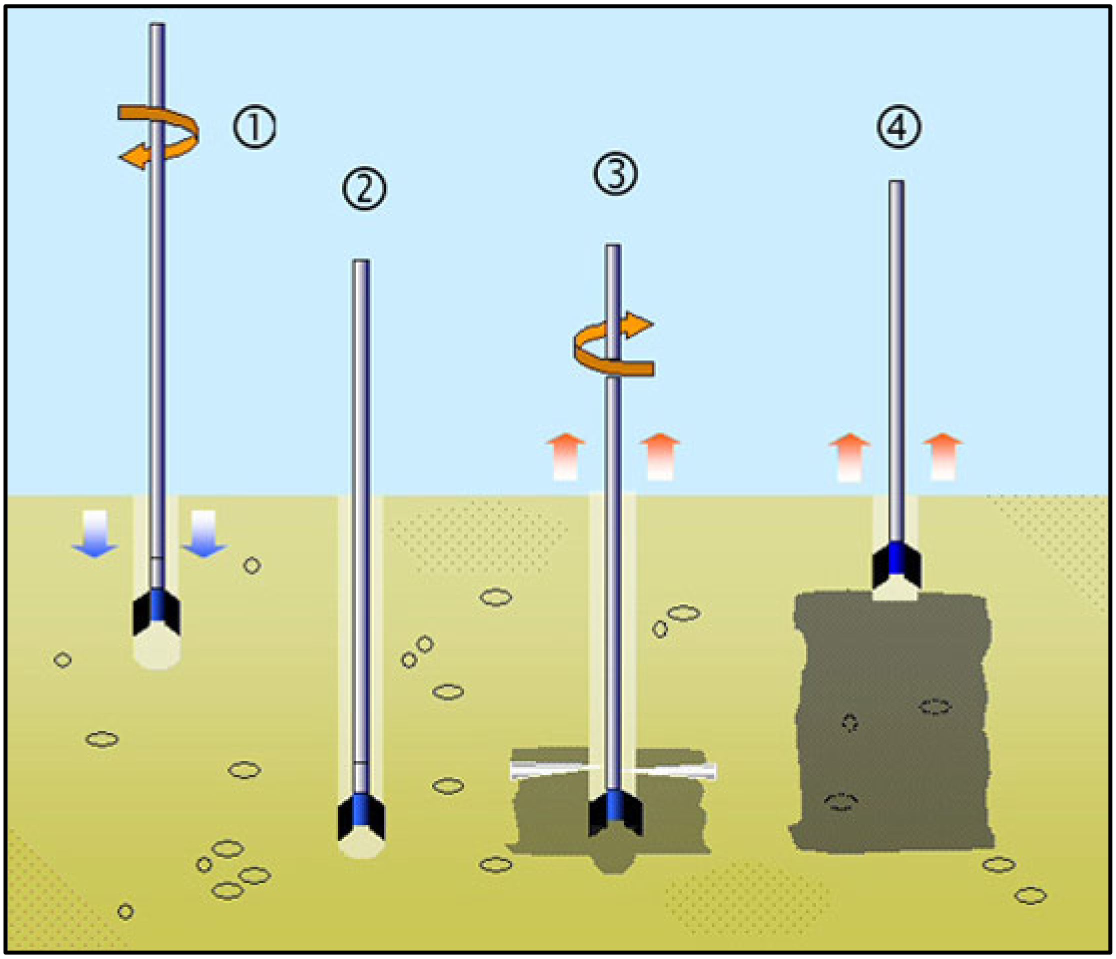


Figure 19-28, Jet Grouting Process Schematic
(Altem İnşaat Ind. ve Trd. Ltd. Corp. (2009))

Jet grouting can be accomplished using 3 different types of jetting procedures as discussed below and depicted in Figure 19-29.

- 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 injecting grout, at a lower pressure, to replace the slurried soil.

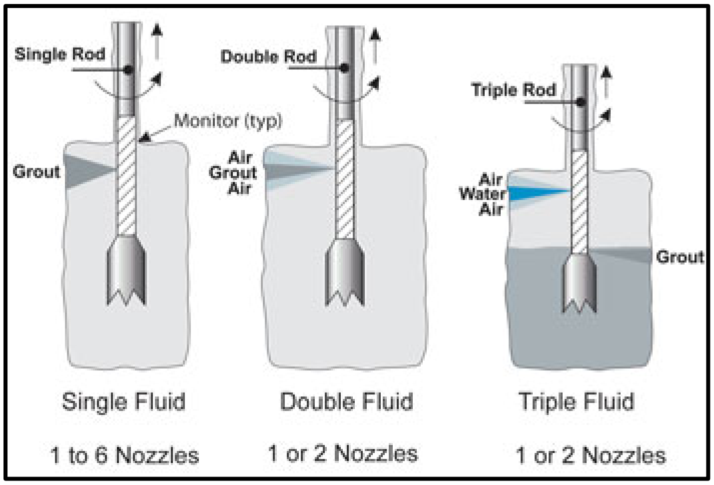


Figure 19-29, Jet Grouting Systems (Burke (Wind Systems) (2010))

19.7.3.5 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 or 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.

19.7.3.6 Slabjacking

Slabjacking is the process of injecting grout under pressure to raise and relevel concrete paving (typically bridge approach slabs) that have settled. 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 Elias, et al. – Vol. II (2006)):

> 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 and have its 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 GEOR 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 COLUMN SUPPORTED EMBANKMENT

Constructing embankments over soft, compressible soils creates numerous problems (i.e., excessive settlements, embankment instability, and long periods over which the settlements 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 alternative ground improvement method has been developed: Column Supported Embankment (CSE) (see Figure 19-30). CSEs consist of 2 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.

In a previous version of the GDM, the Beam Design Approach based on the Modified Collin Method was recommended for use on SCDOT projects regardless of whether rigid (i.e., prestressed concrete, steel H- or pipe piles, or timber) or flexible (i.e., stone, VCCs, DMM, soil mixed or auger cast-in-place piles) columns will be used to transfer the load to the bearing formation. However, because of advances in the design methodology, the Load and Displacement Compatibility (LDC) method will be used for the design of CSEs supported by flexible columns and is described in Schaefer, et al. – Vol. II (2017). Flexible columns require more movement to engage the capacity of the column and therefore, will transfer some of the induced load to the soils located between the columns. The Modified Collin Method will be used for the design of CSEs supported by rigid columns. Rigid columns will not require the amount of movement required to engage the soil as in the flexible columns; therefore, minimal to no load will be transferred to the soil between the columns. The LTP above the rigid columns will act more like a beam or rigid platform then the LTP above flexible columns.

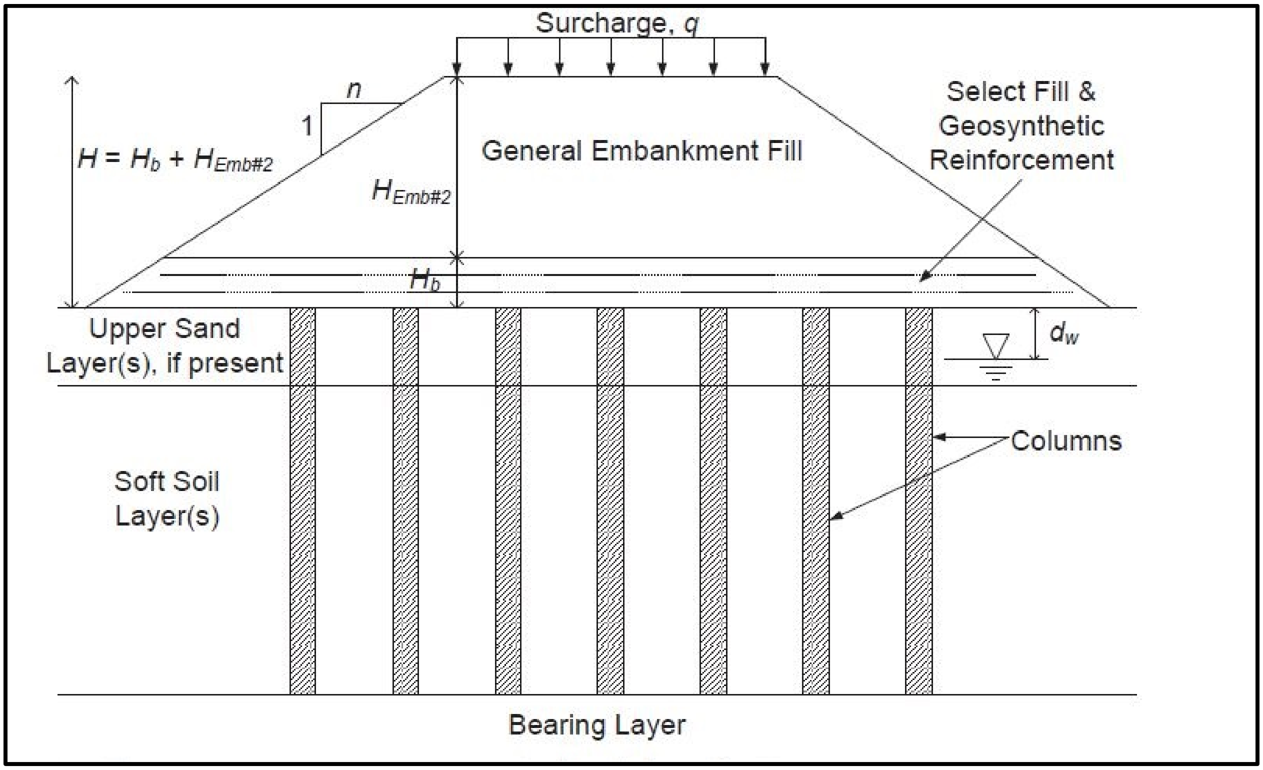


Figure 19-30, CSE with Geosynthetic LTP (Geotech Tools (2012))

The LTP transfers the embankment load to the columns. The LTP may consist of 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. If a rigid transfer platform is required for a project, contact the RPG/SDS and RPG/GDS for guidance. The design of a rigid LTP is the responsibility of the SEOR. The GEOR will provide the nominal resistance of the deep foundation system to be used to support the rigid LTP. The geosynthetic reinforced LTP is discussed in subsequent Sub-sections of this Chapter. The GEOR is responsible for not only designing the columns but also the geosynthetic reinforced LTP.

19.8.1 Analysis and Feasibility

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.

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

19.8.2 Design Approach

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. The Strength limit state failure modes include the following (see Figure 19-31):

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

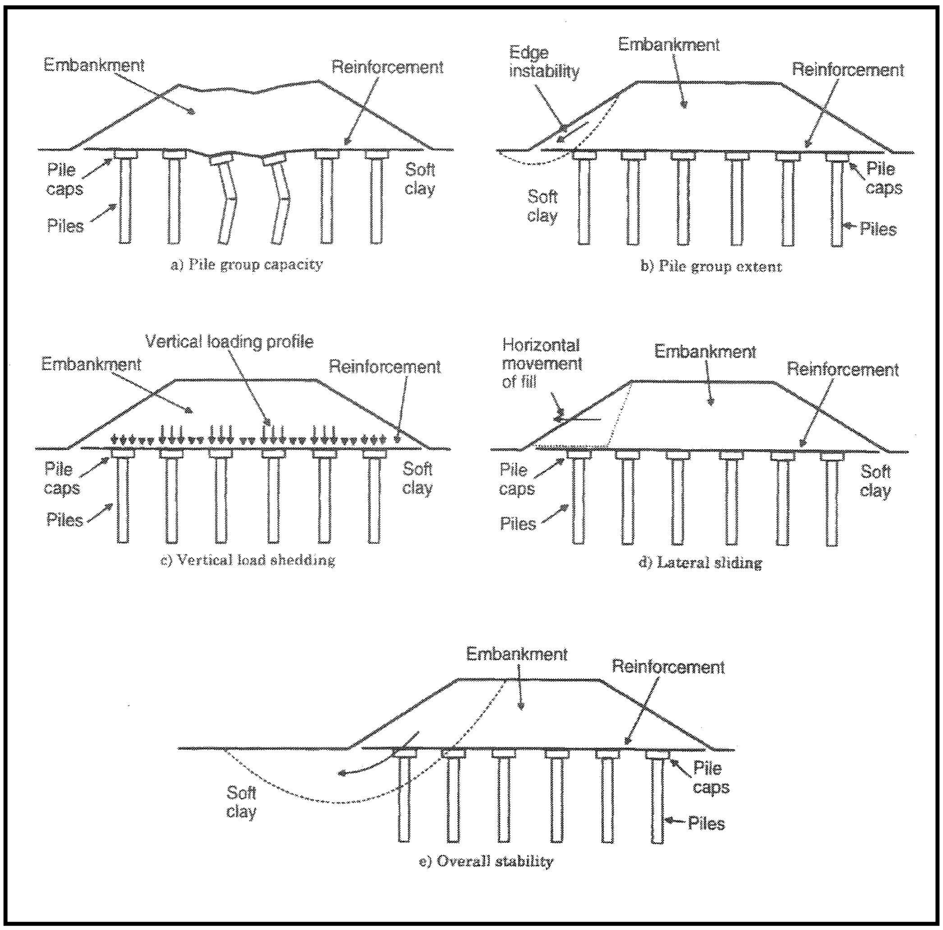


Figure 19-31, Strength Limit State Failure Modes (Schaefer, et al. – Vol. II (2017))

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 19-32, 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 19-32, Detail b).

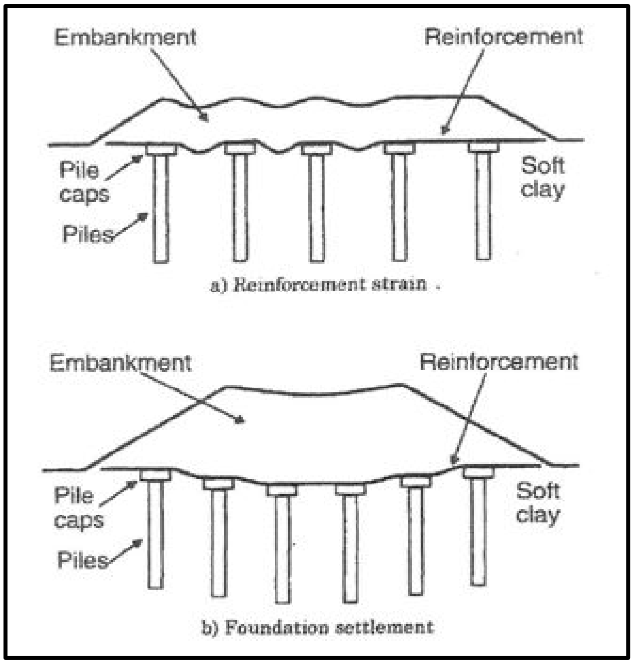


Figure 19-32, Service Limit State
(Schaefer, et al. – Vol. II (2017))

The general design procedure for CSEs is provided below:

1. Estimate preliminary column spacing.
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. Check critical embankment height criteria and adjust column spacing as required.
7. Determine LTP reinforcement requirements based on estimated column spacing. Revise column spacing as required.
8. Determine reinforcement requirements for lateral spreading.
9. Determine overall reinforcement requirements based on LTP and lateral spreading.
10. Check global stability.
11. Prepare construction drawings and specifications.
12. Observe construction.

19.8.2.1 Preliminary Design

The preliminary design of CSEs should consider the following factors:

- The preliminary spacing of the columns should be limited so that the area replacement ratio is between 10 and 20 percent.

- The clear span between columns should be less than the embankment height and should not exceed approximately 10 feet. In addition, the clear span plus twice the column diameter should not exceed the width of the geosynthetic roll, i.e., the geosynthetic should cover 2 rows of columns. 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 for correctly designed and constructed CSEs.

19.8.2.2 Column Design

The selection of the type of column should be based on the constructability, load capacity, and cost of the various column types (Steps 2 and 3 of the general CSE design procedure). The load carrying capacity of each column is based on the tributary area of each column (see Figure 19-33). In CSE design, it is assumed that the weight of the embankment and any surcharge loads are carried by the rigid columns and that the surrounding soil carries no load. For CSEs supported by flexible columns it is assumed that the weight of the embankment and any surcharge loads are carried by both the columns and the surrounding soil. The tributary area for a single column is geometrically a hexagon and is termed a unit cell; however, for simplification a circle having the same tributary area is used. Figure 19-34 provides the effective diameter (D_e) for both equilateral triangular and square spacing. Prior to using rectangular ($s_1 \neq s_2$ in Figure 19-35a) or isosceles triangular column layout contact the OES/GDS for permission. Figure 19-35 provides the determination of the area of the unit cell around each column. 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:

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

Where,

Q_r = Unfactored or nominal column load
D_e = Effective diameter of the tributary area of column or unit cell
H = Height of embankment
γ = Unit weight of embankment soil
q = Live and dead load surcharge (determined similar to long-term stability analysis)

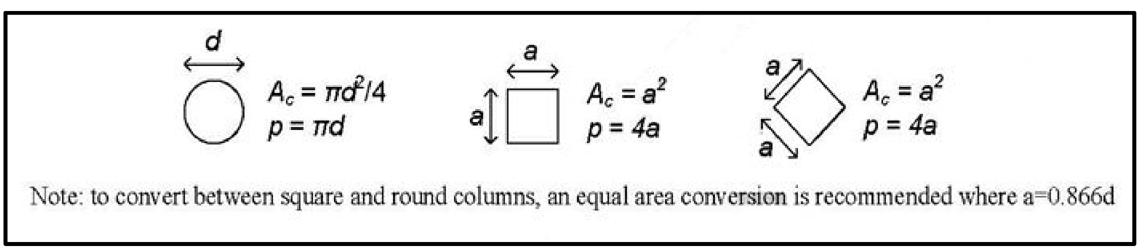


Figure 19-33, Area and Perimeter Determination of Round and Square Columns (Schaefer, et al. – Vol. II (2017))

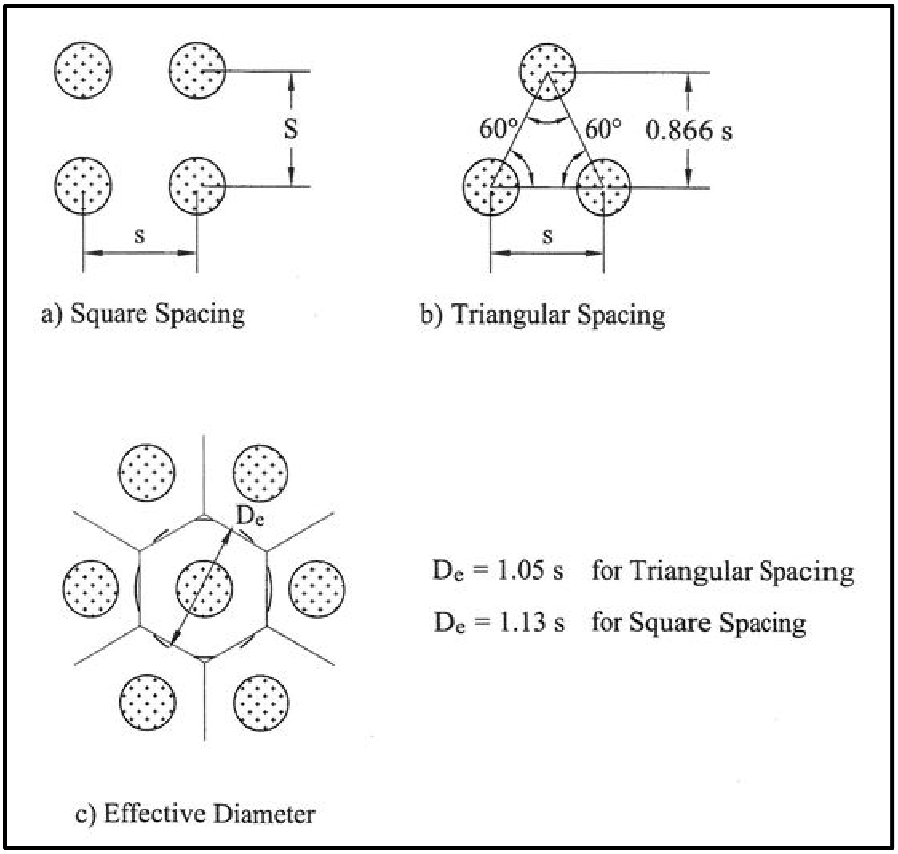


Figure 19-34, Effective Column Diameter Determination (Elias, et al. – Vol. II (2006))

Select the appropriate type of column (i.e., rigid or flexible) based on the Q_r determined previously. For the determination of the resistance of driven concrete, steel, or timber piles, all examples of rigid columns see Chapter 16. For DMM and stone columns (including rammed aggregate piers and VCCs), examples of flexible columns, see previous Sections of this Chapter for the appropriate design methodologies.

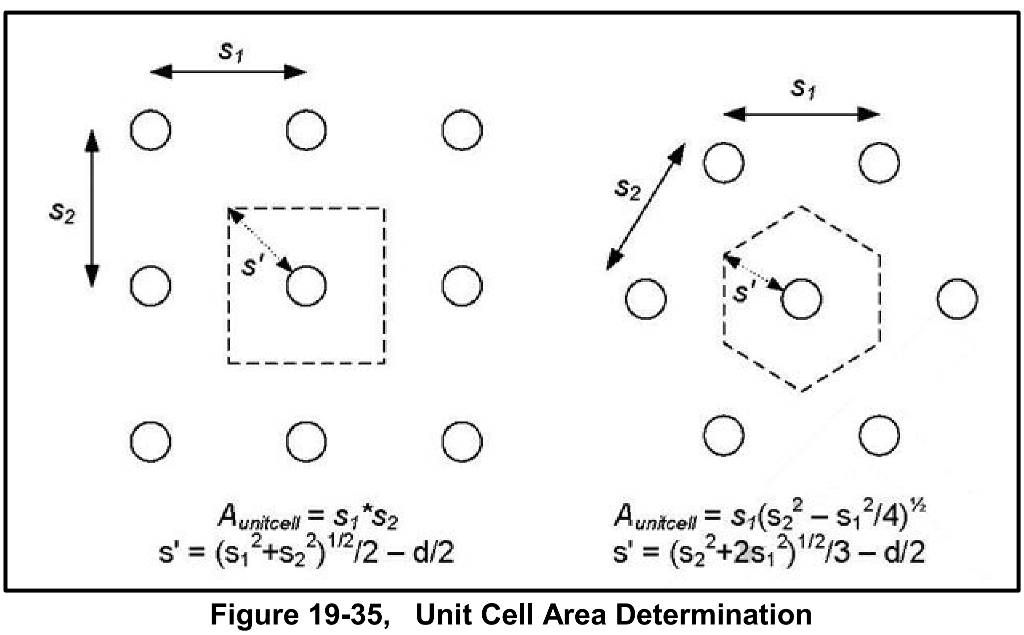


Figure 19-35, Unit Cell Area Determination (Geotech Tools (2012))

19.8.2.3 Lateral Extent of Columns

Step 4 of the general CSE design procedure establishes the lateral extent of the columns. 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 British Standard (BS8006) requires that the columns extend to at least a minimum distance from the proposed toe of slope, L_p, to prevent settlement of the unsupported edge of the embankment from affecting the embankment crest. L_p is determined using the following equations:

$$L_p = H * \left(n - \tan\theta_p\right) \qquad \textbf{Equation 19-26}$$

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

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 19-36)
φ'_{emb} = Effective friction angle of embankment fill

It is typical SCDOT practice for the columns to extend to at least the toe of slope if not 1 row outside of the toe of slope, but within the SCDOT ROW.

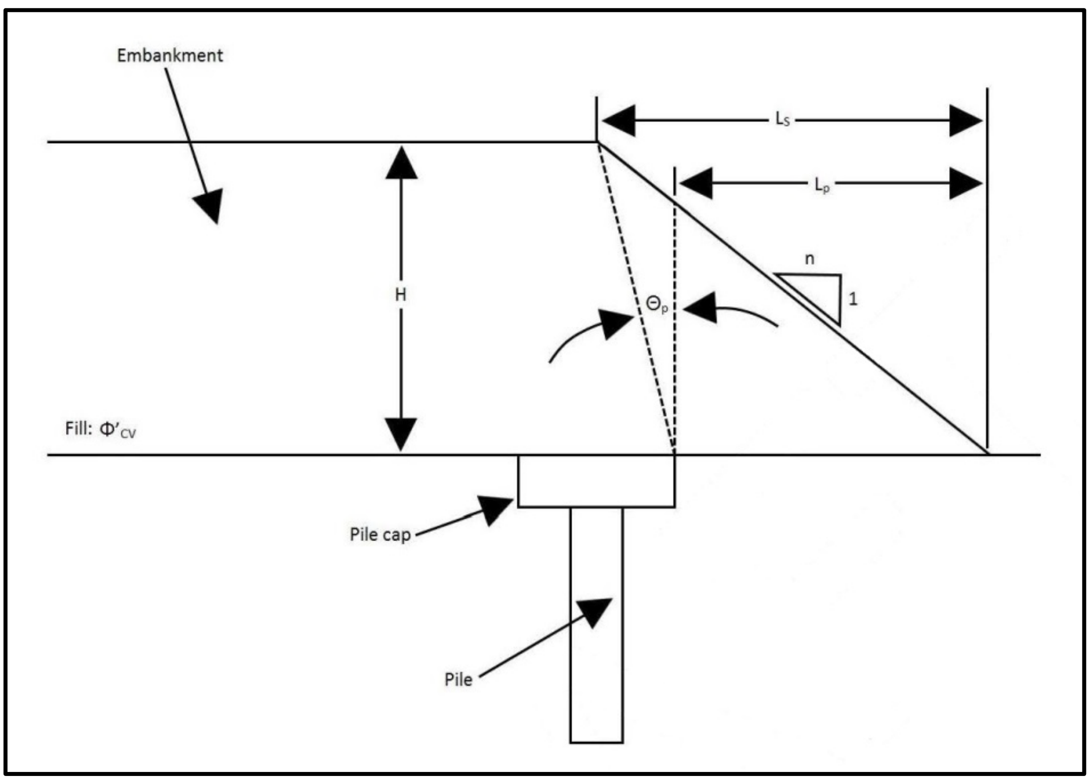


Figure 19-36, CSE Edge Stability
(Schaefer, et al. – Vol. II (2017))

19.8.2.4 Critical Height

According to Schaefer, et al. – Vol. II (2017),

> Avoiding differential settlement at the surface of a CSE is often important, for example, to provide good ride quality and to prevent distress to overlying structures. Factors that influence differential surface settlements include column spacing, column diameter, embankment height, quality of subgrade support relative to column stiffness, and loading acting on the embankment surface. For example, differential surface settlement is likely for a relatively low embankment with wide column spacing and poor subgrade support. Differential surface settlement is unlikely for a high embankment with close column spacing and good subgrade support. In this Chapter, the term *critical height* is defined as the embankment height above which differential settlements at the base of the CSE do not produce measurable differential settlement at the embankment surface.
>
> For CSEs without subgrade support, McGuire (2011) found that the critical embankment height, H_{crit}, depends on the column diameter and spacing, and it is not significantly affected by the relative density of the embankment fill or the use of geosynthetic reinforcement in the load transfer platform…. The approach recommended on www.GeoTechTools.org design document is to use the larger value of H_{crit} estimated…as provided below….

$$H > H_{crit} = max\begin{Bmatrix} 1.5 * (s - a) \\ 1.15 * s' + 1.44d \end{Bmatrix} \quad \textbf{Equation 19-28}$$

Where,

s = Center-to-center distance between columns (see Figure 19-34)
a = Face dimension for a square column (see Figure 19-33)
s’ = Determined in Figure 19-35
d = Diameter for a round column (see Figure 19-33)

> In cases where a square array of *either* square pile caps *or square piles without caps* is used and the embankment height is fixed by the difference between the embankment subgrade elevation and roadway elevation, the minimum center-to-center spacing can be estimated by Equation *19-29*. If *a square array of round* pile caps *or round piles without caps is used*, 0.866d can be substituted for a pile cap width, a, *resulted* in *Equation 19-30.*

$$s \leq 1.2 * (H - a) \quad \textbf{Equation 19-29}$$

$$s \leq 1.2 * (H - 0.866d) \quad \textbf{Equation 19-30}$$

If an equilateral triangular spacing is used for either square pile caps or square piles without caps use Equation 19-31. If round pile caps or round piles without caps are placed in an equilateral triangular array use Equation 19-32 to determine the required spacing.

$$s \leq 1.5 * (H - a) \quad \textbf{Equation 19-31}$$

$$s \leq 1.5 * (H - 0.866d) \qquad \text{Equation 19-32}$$

19.8.2.5 Load Transfer Platform

The design of the LTP shall be based on whether rigid or flexible columns are being used to support the LTP. If rigid columns are being used to support the CSE, then the LTP shall be designed using the beam method, specifically the Modified Collin Method. For flexible columns used to support the CSE, the LTP shall be designed using the LDC method.

19.8.2.5.1 Modified Collin Method

The Modified Collin Method or the beam design approach shall be used when rigid columns and a geosynthetic reinforced LTP are used together. As indicated previously the rigid columns may be prestressed concrete piles, steel H-piles, pipe piles, or timber piles. 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 3 layers of reinforcement (Figure 19-37). In addition, in the Modified Collin Method, a catenary reinforcement is added at the base of the beam to support the soil beneath the arch.

The Modified Collin Method is based on the following assumptions:

- The thickness (h) of the LTP is equal to or greater than ½ of the clear span between the columns (i.e., 0.5(s-d))
- A minimum of 3 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 with an effective friction angle greater than or equal to 35°
- 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 5 percent

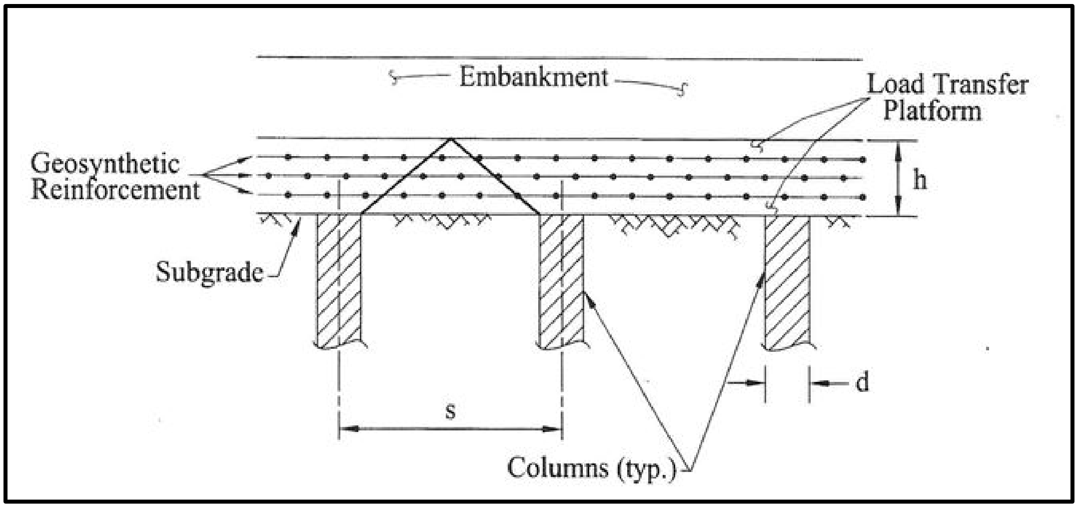


Figure 19-37, Load Transfer Platform
(Elias, et al. – Vol. II (2006))

The fill load attributed to each layer of reinforcement is the material located between the layer of reinforcement and the next layer above (Figure 19-38). 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.

$$W_{Tn} = \frac{\left[(s-d)_n^2+(s-d)_{n+1}^2\right]*\sin 60°*h_n*\gamma_{emb}}{(s-d)_n^2*\sin 60°} \quad \textbf{Equation 19-33}$$

$$W_{Tn} = \frac{\left[(s-d)_n^2+(s-d)_{n+1}^2\right]*h_n*\gamma_{emb}}{(s-d)_n^2} \quad \textbf{Equation 19-34}$$

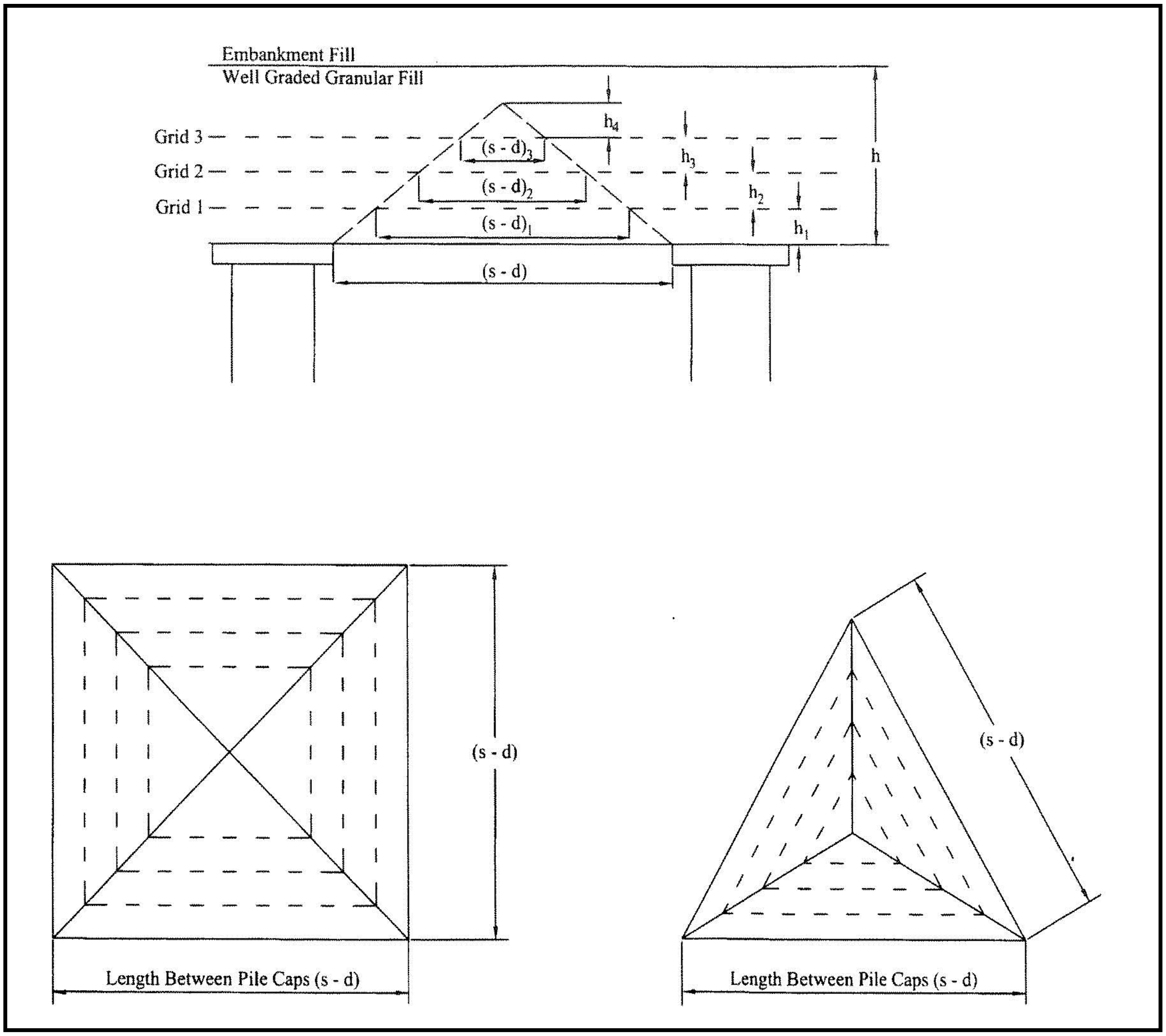


Figure 19-38, Collin Method Load Transfer Platform Design (Schaefer, et al. – Vol. II (2017))

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:

$$T_{RPn} = \frac{W_{Tn} * \Omega * D}{2}$$ **Equation 19-35**

Where,

D = $(s\text{-}d)_n$ for square column spacing

D = $(s\text{-}d)_n \tan 30°$ for triangular column spacing

Ω = From Table 19-28

Table 19-28, Values of Ω
(Schaefer, et al. – Vol. II (2017))

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

According to Elias, et al. – Vol. II (2006):

> Based on research recently completed (*Collin, Han and Huang (2005)*) using numerical modeling, the *Collin Method* has been modified. The modification involves the addition of 1 layer of reinforcement at *the* subgrade. This layer of reinforcement is designed as a catenary to carry the load from the soil below the arch (Figure *19-39*).

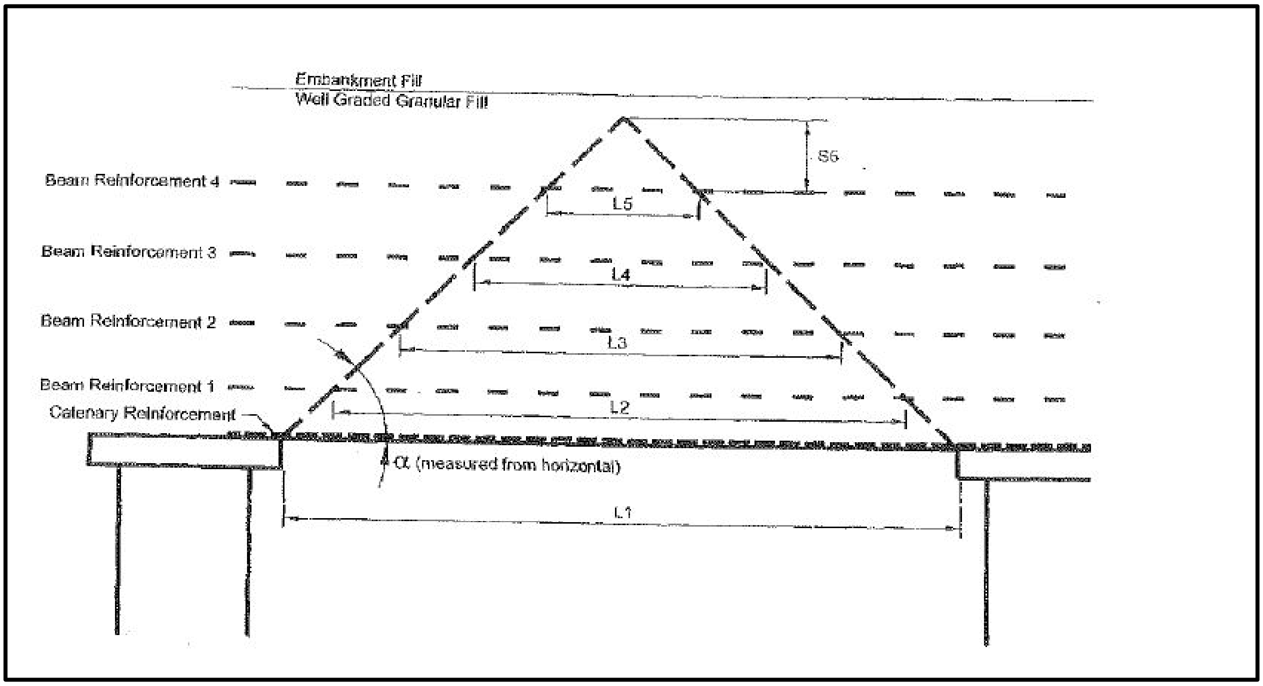


Figure 19-39, Modified Collin Method Reinforcement
(Elias, et al. – Vol. II (2006))

The uniform vertical load on the catenary layer of reinforcement (W_{TC}) may be determined from the equation below *(which is applicable to either square or triangular column spacing)*:

$$\boldsymbol{W_{TC}} = \left(\frac{\sum_1^n \boldsymbol{h} * \boldsymbol{\gamma}}{\mathbf{3}}\right) \qquad \textbf{Equation 19-36}$$

Where,

Σh = Total height of arch (see Figure 19-38)

γ = *Unit weight of LTP (beam) material*

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

$$T_{RPC} = \frac{W_{TC} * \Omega * D}{2}$$ **Equation 19-37**

Where,

D = Design span for tensioned membrane

Square column layout

$$D = 1.41 * \left[(s-d) - 2 * \left(\frac{\sum Vertical\ Spacing}{tan\,45°}\right)\right]$$ **Equation 19-38**

Triangular column layout

$$D = 10.867 * \left[(s-d) - 2 * \left(\frac{\sum Vertical\ Spacing}{tan\,45°}\right)\right]$$ **Equation 19-39**

Ω = Dimensionless factor from tensioned membrane theory (*Table 19-28*)

19.8.2.5.2 Load and Displacement Compatibility Method

The Load and Displacement Compatibility (LDC) Method shall be used if flexible columns will be used to support the CSE. Flexible columns consist of stone columns, VCCs, DMM columns, soil mixed columns, or auger cast-in-place piles. Schaefer, et al. – Vol. II (2017) recommends using the LDC method, stating:

> In order for the CSE design to be effective, the embankment load must be transferred to the columns without excessive deformations occurring at the surface of the embankment. … A practical method that models the actual load transfer mechanisms is the LDC method.
>
> Smith (2005) and Filz and Smith (2006, 2007) developed a *LDC* method for analyzing the net vertical load that acts on the geosynthetic reinforcement in the LTP.…Essential features of the LDC method include:
>
> - Vertical load equilibrium and displacement compatibility are assumed at the level of the geosynthetic reinforcement to calculate the load distribution amount the columns, the soft soil between columns, the geosynthetic, and the base of the embankment above columns and between columns.
> - An axisymmetric approximation of a unit cell is employed for calculating the vertical load acting on the geosynthetic reinforcement.
> - A 3D representation of the geosynthetic-reinforced CSE system and a parabolic deformation pattern of the geosynthetic between adjacent columns is assumed for the purpose of calculating the tension in the geosynthetic *(i.e., Generalized Parabolic Method)*.
> - The LDC method was developed for round columns or square pile caps in a square array.
> - Nonlinear response of the embankment is incorporated by providing linear response up to a limit state, at which point additional base settlement

produces no further load concentration on the columns. The limit state is determined using the Adapted Terzaghi Method described below.

- Linear stress-strain response of the geosynthetic is assumed, but because large displacements of the geosynthetic are involved, the load-displacement relationship for the geosynthetic is nonlinear. Iterations can be performed to approximate nonlinear response of the geosynthetic material.
- Nonlinear compressibility of *Clay-Like* soil between columns is represented using the compression ratio *(C_c or $C_{\varepsilon c}$)*, recompression ratio *(C_r or $C_{\varepsilon r}$)*, and preconsolidation pressure *(σ'_p or p'_c)*.
- Slippage is allowed between the soil and the column when the interface shear strength is exceeded.

An exploded profile view of a unit cell, including the vertical stresses at the contacts above and below the geosynthetic reinforcement is shown in *Figure 19-40*. Vertical equilibrium of the system shown in *Figure 19-40* is satisfied when:

$$\gamma * H + q = \Xi_{geotop} = \Xi_{geobot}$$ **Equation 19-40**

$$\boldsymbol{\Xi_{geotop} = a_s * \sigma_{col,geotop} + (1 - a_s) * \sigma_{soil,geotop}}$$ **Equation 19-41**

$$\boldsymbol{\Xi_{geobot} = a_s * \sigma_{col,geobot} + (1 - a_s) * \sigma_{soil,geobot}}$$ **Equation 19-42**

$$\boldsymbol{a_s = \frac{A_c}{A_{unitcell}}}$$ **Equation 19-43**

Where,

γ = Unit weight of embankment soil
H = Height of embankment
q = Surcharge pressure
a_s = Area replacement ratio
A_c = Area of column or pile cap
$A_{unitcell}$ = Area of unit cell (see Figure 19-37)
$\sigma_{col,geotop}$ = Average vertical stress acting downward at the top of geosynthetic in the area underlain by the column (see Figure 19-42)
$\sigma_{soil,geotop}$ = Average vertical stress acting downward at the top of geosynthetic in the area underlain by the soil foundation (see Figure 19-42)
$\sigma_{col,geobot}$ = Average vertical stress acting downward at the bottom of geosynthetic in the area underlain by the column (see Figure 19-42)
$\sigma_{soil,geobot}$ = Average vertical stress acting downward at the bottom of geosynthetic in the area underlain by the soil foundation (see Figure 19-42)

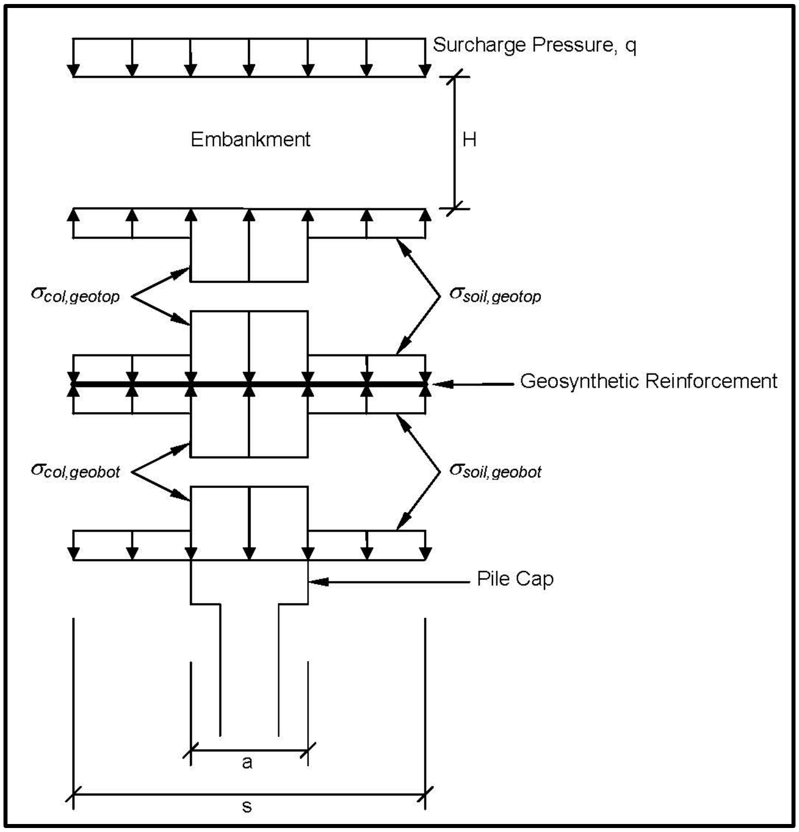


Figure 19-40, Definition Sketch for LDC Method (Schaefer, et al. – Vol. II (2017))

Load-deflection relationships were developed for:

i. The embankment settling down around the column or pile cap;
ii. The geosynthetic deflecting down under the net vertical load acting on the area underlain by soil; and
iii. The soil settling down between the columns.

The relationships are only described in conceptual terms here; however, supporting equations and additional details are presented by Filz and Smith (2006). The composite foundation system consisting of columns and the soil between the columns is discretized, and the simultaneous nonlinear equations can be solved numerically using a spreadsheet program.

The load-deflection relationship for the embankment settling down around the column or pile cap *(i above)* is assumed to be linear up to the maximum load condition. The linear part is approximated using a linear solution for displacement of a circular loaded area on a semi-infinite mass. As indicated previously, square pile caps of width, a, can be approximated as circular pile caps with diameter, d, such that the pile cap areas are the same (a = 0.866d). The limiting stress condition in the embankment above the geosynthetic reinforcement is established using the Adapted Terzaghi Method with a lateral earth pressure coefficient, K, of 0.75, which is between the values of 1.0 used by Russell and Pierpoint (1997) and 0.5 used by Russell, Naughton and Kempton (2003).

The geosynthetic deflects down under the net vertical load applied over the area underlain by soil *(ii above)*. The geosynthetic load-deflection relationship was developed based on analyses of a uniformly loaded annulus of linear elastic membrane material with the inner boundary pinned, which represents the support provided by the column, and with the out boundary free to move vertically but not laterally, which represents the axisymmetric approximation of lines of symmetry in the actual 3-dimensional configuration of a column-supported embankment. The details of the analyses and the results are presented by Smith (2005) and Filz and Smith (2006).

The settlements of the column and the subgrade soil are determined based on the vertical stress applied to the top of the column or pile, $\sigma_{col,geobot}$, and the vertical stress applied to the subgrade soil, $\sigma_{soil,geobot}$ *(iii above)*. The column compression is calculated based on a constant value of the column modulus. One-dimensional compression of *Clay-Like* soil located between columns is calculated using the compression ratio *(C_c or $C_{\varepsilon c}$)*, recompression ratio *(C_r or $C_{\varepsilon r}$)*, and preconsolidation pressure *(σ'_p or p'_c)* of the soil. If an upper layer of *Sand-Like soil* is located between the columns, the *Sand-Like soil* compression is calculated using a constant value modulus for the *Sand-Like soil*. If voids are anticipated between the LTP and subgrade soil the support from the foundation should be ignored.

As the compressible soil settles down with respect to the stiffer column, the soil sheds load to the column through shear stresses at the contact between the soil and the column along the column perimeter. The magnitude of the shear stress is determined using an effective stress analysis and a value of interface friction angle between the soil and column. The vertical stress increment in the soil from the embankment, and surcharge loads, decreases with depth due to the load shedding process until the depth at which the column settlement and soil settlement are equal. An important detail is that the settlement profile of the subgrade soil at the level of the top of the columns is likely to be dish-shaped between the columns. The difference between the column compression and the average soil compression is the average differential settlement at the subgrade level. To account for the dish-shaped settlement profile between the columns, the suggestion by Russell, et al. (2003) that the maximum differential settlement at subgrade level may be as much as twice the average differential settlement was adopted.

The computational method described above is solved by satisfying vertical equilibrium using *Equations 19-40 to 19-43* and requiring that the calculated values of the differential settlement at subgrade level must be the same for the base of the embankments, the geosynthetics if utilized, and the underlying foundation soil. If there is reason to believe that the soft soil between the columns will settle more than the geosynthetic deforms, e.g., due to groundwater lowering, then the subgrade soil can be assigned a very high compressibility value to essentially eliminate subgrade support of the geosynthetic. The simultaneous nonlinear equations that describe this computational method have been implemented in a spreadsheet **Geogrid Bridge 2.0** (Filz and Smith (2006)) *that is available for purchase (see CGPR #77) at the following website*:

http://www.cgpr.cee.vt.edu/index.php?do=searchpublication&keyword=*.

GeogridBridge 2.0 has the following features:

- Two different types of embankment fill are allowed so that lower quality fill can be used above the bridging layer.
- Analyses without geosynthetic reinforcement can be performed by setting the value of the geosynthetic stiffness, J, equal to 0.
- The column area and properties can vary with depth so that embankments supported on piles with pile caps can be analyzed.
- The subsurface profile can include 2 upper sand layers and 2 underlying clay layers. The preconsolidation stress for the clay can vary linearly within each clay layer.
- The simultaneous nonlinear equations are solved automatically, and the input and output are arranged so that design alternatives can be evaluated easily.

The LDC method was validated by comparison with numerical analyses that were previously validated by comparison with instrumented case histories and pilot-scale experiments performed by others.

19.8.2.6 Lateral Spreading

The potential for lateral spreading of the embankment must be analyzed (Figure 19-41). 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 (P_{Lat}) is determined using the following equations.

$$\boldsymbol{P_{Lat} = K_a * \left(\frac{\gamma * H^2}{2} + q * H\right)} \qquad \textbf{Equation 19-44}$$

Sand-Like (≤ 20 percent fines)

$$\boldsymbol{K_a = tan^2\left(45 - \frac{\phi_{emb}{}'}{2}\right)} \qquad \textbf{Equation 19-45}$$

Sand-Like (> 20 percent fines, PI ≤ 10)

$$\boldsymbol{K_a = tan^2\left(45 - \frac{\phi_{emb}{}'}{2}\right) - \frac{2c_{emb}{}'}{\sigma_v'} * \left[tan^2\left(45 - \frac{\phi_{emb}{}'}{2}\right)\right]} \qquad \textbf{Equation 19-46}$$

Where,

φ'_{emb} = Effective friction angle of embankment fill
c'_{emb} = Effective cohesion of embankment fill
σ'_v = Effective overburden pressure at bottom of embankment fill

The resistance to lateral spread without geosynthetic reinforcement is determined by:

$$R_{Ls} = (L_s) * S_u \quad \textbf{Equation 19-47}$$

Where,

L_s = Length of side slope of the embankment
S_u = Undrained shear strength of foundation soil

P_{Lat} is compared to R_{ls} and shall have an φ less than or equal to 0.66 as indicated in the following equation:

$$\left(\frac{P_{Lat}}{R_{ls}}\right) \leq 0.66 \quad \textbf{Equation 19-48}$$

If an adequate φ cannot be achieved, geosynthetic reinforcement shall be added. The reinforcement shall be able to resist all of P_{Lat}. The reinforcement long-term design strength (T_{al}) shall be greater than P_{Lat}. Multiple layers of reinforcement may be used to resist the lateral spreading force. The geosynthetic reinforcement materials to be used to resist lateral spreading shall meet the criteria provided in the latest version of the STSs for *Geogrid Soil Reinforcement*, SC-M-203-2 or *Geotextile Soil Reinforcement*, SC-M-203-3.

$$T_{al} \geq P_{Lat} \quad \textbf{Equation 19-49}$$

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

$$L_e = \frac{T_{al}}{\left[0.5\gamma * H * \left(c_{iemb} * \tan \phi'_{emb}\right)\right]} \quad \textbf{Equation 19-50}$$

Where,

c_{iemb} = Coefficient of interaction for sliding between the geosynthetic reinforcement and the embankment fill
φ'_{emb} = Friction angle of embankment fill material

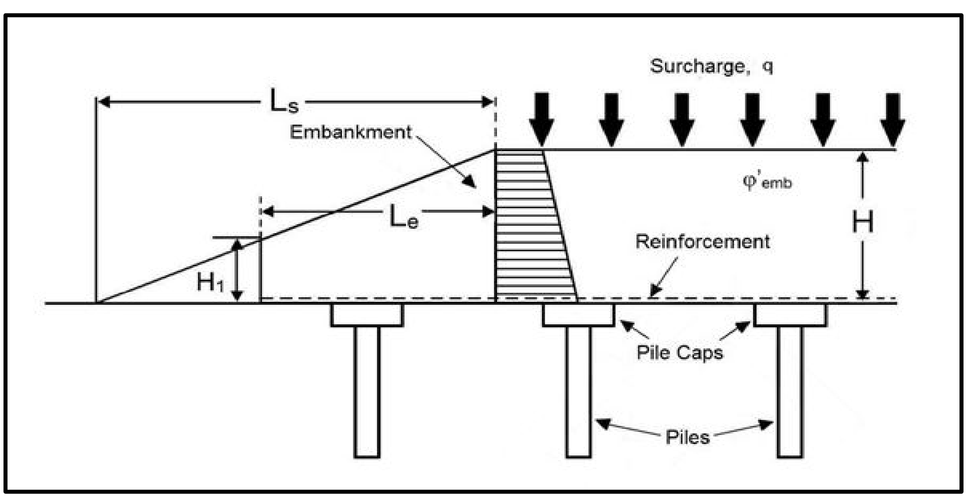


Figure 19-41, CSE Lateral Spreading (modified Schaefer, et al. – Vol. II (2017))

19.8.3 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)

$$T_{max} = T_{RP} \qquad \textbf{Equation 19-51}$$

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

$$T_{max} = T_{RP} + T_{ls} \qquad \textbf{Equation 19-52}$$

19.9 CONSTRUCTION WORKING PLATFORM

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. The design requirements for reinforced embankments and RSSs are provided in Chapter 17. This Section handles the design of construction working platforms that are intended as an aide to construction.

Please note that the following information is obtained from Holtz, Christopher and Berg (2008) instead of Elias, et al. – Vol. II (2006). The Elias, et al. – Vol. II (2006) Chapter that covers this topic is actually a draft Chapter from Holtz, et al. (2008). Therefore, SCDOT has elected to use the Holtz, et al. (2008) instead.

The use of a combination of stone or granular materials and geosynthetic 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., contains 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 bridge lift 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 reinforcement used in subgrade stabilization is not included in the stability analysis. 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

Muck excavation or undercut should be limited to no more than 5 feet. If a suitable bearing soil is not encountered within this depth or unless otherwise specified by the GEOR, a geosynthetic material meeting the requirements of Supplement Technical Specification (STS) *Geosynthetic Materials for Separation and Stabilization* (SC-M-203-1) shall be placed beneath the bridge lift material. The use of a geogrid separator should be considered for Sand-Like subsoils and a geotextile separator should be considered for Clay-Like subsoils. Place the geosynthetic material in the bottom of the excavation and up the excavation side slopes. In areas that require muck excavation or undercutting, replace with bridge lift material.

Borrow excavation materials and man-made (lightweight aggregates) may be placed as bridge lift materials as long as the grade on which the material is being placed is at least 6 inches above ground water level. Borrow excavation materials bridge lift materials shall have a maximum lift thickness of 1 foot. In the event that groundwater does not allow backfilling with a borrow excavation material, place either a stone or granular material as the bridge lift material that meets the Supplemental Specification *Bridge Lift Materials*. Bridge lift materials placed in water shall consist of either stone or coarse granular materials (A-1-a).

Stone bridge lift materials shall have maximum lift thicknesses of 2 feet and shall extend a minimum of 6 inches above the water level surface. Stone bridge lift materials shall not be placed through more than 5 feet of water. For placement of materials through water depths greater than 5 feet see Chapter 17. Granular lift materials shall also have a lift thickness of 1-1/2 feet and shall not be placed in more than 2 feet of water. Granular bridge lift materials shall extend a minimum of 2 feet above water level surface. Individual bridge lifts shall have some type of limited compactive/tamping effort. If additional compacted borrow excavation soil is needed to reach grade, a geosynthetic material meeting the requirements of STS *Geosynthetic Materials for Separation and Stabilization* (SC-M-203-1) shall be placed between any stone bridge lift material and the overlying compacted soil. Bridge lifts consisting of either borrow excavation or granular bridge lift material shall not be placed within 3 feet of the base of the pavement section. Only compacted borrow excavation soil or stone bridge lift material shall be placed within this zone.

The thickness of the bridge lift is determined using both the US Forest Service (Steward, Williamson and Mohney (1977)) and the Giroud and Han (2004a and b) (also called Giroud-Han) methods as presented in Holtz, et al. (2008). 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. If the US Forest Service method is used to determine the depth of undercutting and the thickness of the bridge lift, then the maximum size of construction equipment shall be indicated on the plans (see Figures 19-42 and 19-43).

19.9.1 <u>US Forest Service (Steward, et al. (1977)) Method</u>

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 FVSTs. Undrained shear strength, in psi, may also be estimated from field CBR values using the following equation.

$$\boldsymbol{c_u = 4.3 * (CBR)}$$ **Equation 19-53**

The second and third steps handle the anticipated traffic configuration. The type of construction equipment anticipated should be indicated as well as the amount of 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 conditions: without reinforcement and with reinforcement. The table below provides the bearing capacity factor based on the reinforcement condition, tolerable rut depth, and traffic.

Table 19-29, Bearing Capacity Factors for USFS Method (adopted from Holtz, et al. (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 6 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 19-42 for single wheel loads and 19-43 for dual wheel loads depending on the vehicular configuration assumed in the third step.

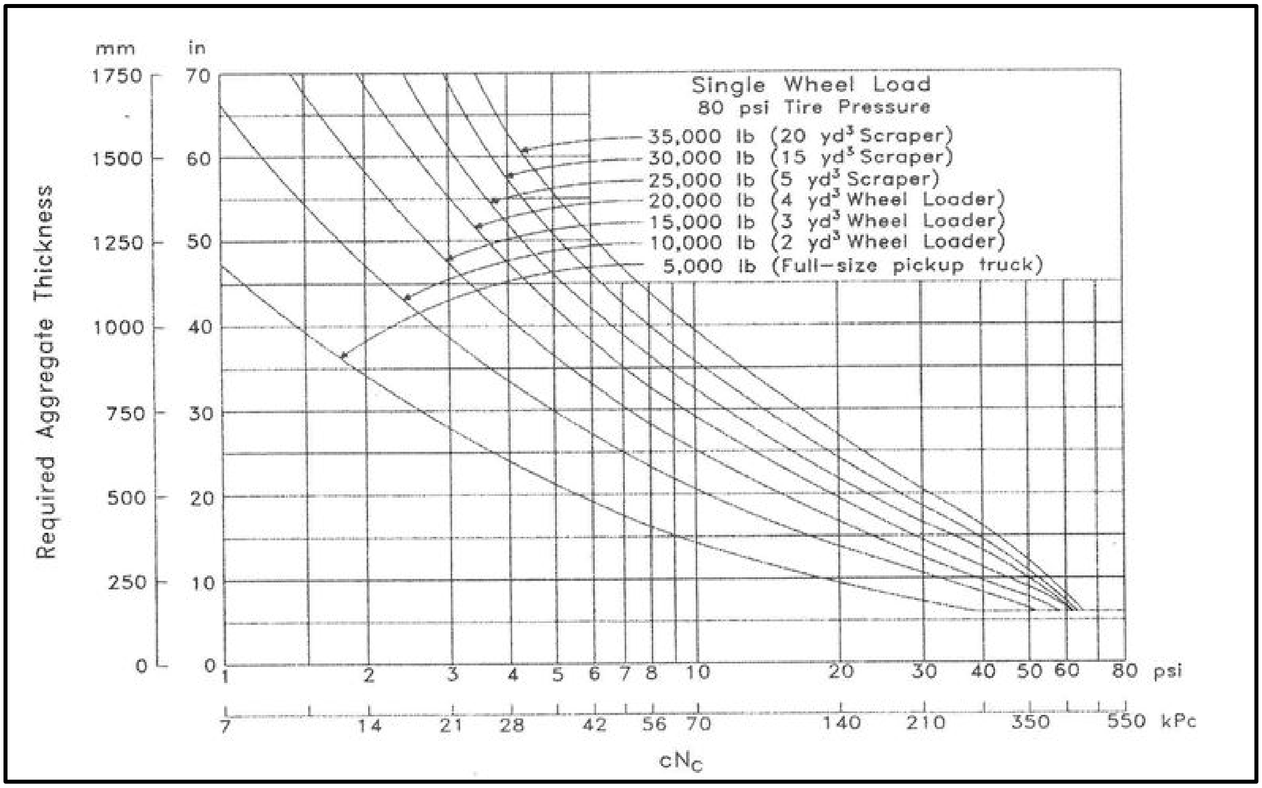


Figure 19-42, USFS Method Bridge Lift Thickness – Single Wheel Loads (adopted from Holtz, et al. (2008))

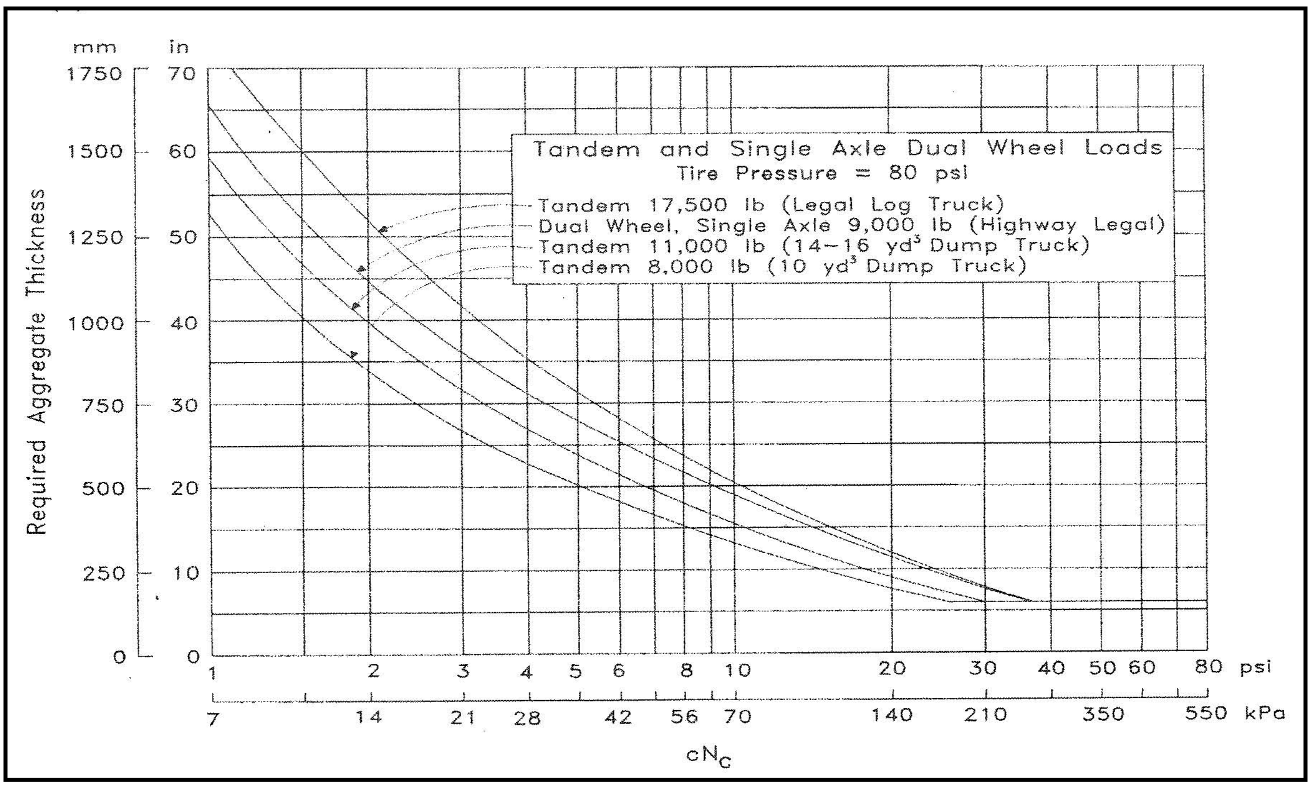


Figure 19-43, USFS Method Bridge Lift Thickness – Dual Wheel Loads (adopted from Holtz, et al. (2008))

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 3. 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 to 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.

19.9.2 Giroud-Han Method

As indicated previously, the Giroud-Han method is based on Giroud and Han (2004a and b). The Giroud-Han method is an iterative process since the required thickness of bridging material is on both sides of the following equation:

$$\boldsymbol{h = \frac{0.868+4*(Y)*(X)^{1.5}}{1.816} * \left(\sqrt{\frac{80}{\frac{s}{3}*\left(1-0.9e^{-X^2}\right)*N_c*c_u}} - 1\right) * 6.3} \quad \textbf{Equation 19-54}$$

Where,

$$\boldsymbol{Y = (0.661 - 1.006 * J^2) > 0.0} \quad \textbf{Equation 19-55}$$

$$\boldsymbol{X = \frac{r}{h} = \frac{6.3}{h}} \quad \textbf{Equation 19-56}$$

Where,

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

Table 19-30, Bearing Capacity Factor and Aperture Stability Modulus (adopted from Holtz, et al. (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 19-55, but reported in N-m/degree)

The following assumptions and limitations are placed on the Giroud-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.

$$P_{h=0,unreinf} = \left(\frac{s}{3}\right) * 391.53 * c_u \qquad \textbf{Equation 19-57}$$

Where,

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

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 19-54 should be used to determine the required thickness of bridge lift. Utilizing a P of 10,000 pounds in Equation 19-57, the minimum undrained shear strength with corresponding rut depth is shown in the following table.

Table 19-31, Minimum Undrained Shear Strength versus Rut Depth

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

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