CHAPTER 14

GROUND IMPROVEMENT TECHNOLOGY
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14.1 OVERVIEW

At sites where an inadequately performing subsurface material may negatively impact the stability, function, or construction of a structure or system, *Ground Improvement Technology* may be utilized as a treatment for the unacceptable subsurface conditions. Ground Improvement Technology serves to improve or stabilize the subsurface material’s performance or to improve the load conditions applied to the unsatisfactory subsurface material. Ground Improvement Technology differs from other site improvement methods, such as retaining structures or deep foundations, in that the unsatisfactory subsurface conditions at a site are addressed by enhancing the performance of the in-situ subsurface materials.

According to *Ground Improvement Methods*, Volume I, FHWA NHI-06-019, August 2006, “One of the major functions of geotechnical engineering is to design, implement and evaluate ground improvement schemes for infrastructure projects. During the last 25 years significant new technologies and methods have been developed and implemented to assist the geotechnical specialist in providing cost-effective solutions for construction on marginal or difficult sites.” The ground improvement methods discussed in this Chapter are based on the contents of *Ground Improvement Methods*, Volumes I and II but the concepts presented should not be viewed as the complete discussion of ground improvement methods since new approaches or technologies are always being evaluated. For simplicity within this chapter, *Ground Improvement Methods*, Volumes I and II, FHWA NHI-06-019 and FHWA NHI-06-020, August 2006 will be referenced as *Ground Improvement Methods*. The Departmental Geotechnical Engineer should consult each volume for more details concerning a specific ground improvement method.

The Departmental Geotechnical Engineer should endeavor to be aware of new and innovative ground improvement ideas. If a new or innovative ground improvement method is to be used on a NYSDOT project, approval must be first obtained from the Geotechnical Engineering Bureau. The approval process will consist of an evaluation on the engineering design, the desired outcome and verification procedure, construction methodology, and availability of construction experience/contractors to perform the specified type of work.

Ground improvement methods are used to stabilize or enhance the performance of poor/unsuitable subsurface soils and/or to augment the performance of embankments, structures, or subsurface systems. Generally, these methods are used when replacement of the unacceptable, in-situ soils is impractical, or is too costly. Ground improvement methodologies are designed to enhance one or more of the following primary functions within the inadequately performing soil or rock:

**Subsurface Function Enhancements Achieved By Ground Improvement Technology:**

- Increase or stabilize bearing capacity, or shear strength,
- Limit and control non-uniform or excessive surface deformations,
- Accelerate primary consolidation,
- Decrease long-term, total settlement,
• Provide/increase lateral stability,
• Provide seepage cutoffs or control or minimize amounts of detrimental voids,
• Increase resistance to liquefaction, and
• Improve stability during dynamic loading.

According to Ground Improvement Methods, at least one of three general strategies is utilized to accomplish the above functions:

General Strategies For Enhancing Subsurface Functions:

1. Alter the soil or rock mass properties by increasing or stabilizing its shear strength, density, and/or by decreasing its compressibility or seepage potential,
2. Use lightweight fills to significantly reduce the applied load on the inadequately performing soil, and/or
3. Transfer a portion of the applied load to a more competent, deeper, subsurface material.

14.1.1 Process for Identifying Appropriate Methods

Ground Improvement Methods recommends a sequential design process that includes a sequence of evaluations that proceed from simple to more detailed. This process helps identify the most appropriate ground improvement method(s) and is described in Table 14-1.

<table>
<thead>
<tr>
<th>Step</th>
<th>Process</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>Perform subsurface investigations at the project site. Identify potential poor ground conditions, including extent and type of negative impact. Identify variables causing the unacceptable subsurface conditions.</td>
</tr>
<tr>
<td>2</td>
<td>Identify or establish performance requirements of the unacceptable subsurface material.</td>
</tr>
<tr>
<td>3</td>
<td>Identify and assess any space, height, or environmental constraints.</td>
</tr>
<tr>
<td>4</td>
<td>Assessment of subsurface conditions – type, depth and extent of poor soil as well as groundwater table depth and assessment of shear strength and compressibility potential.</td>
</tr>
<tr>
<td>5</td>
<td>Preliminary selection of ground improvement method(s) – takes into account performance criteria, limitations imposed by subsurface conditions, schedule and site or environmental constraints, and amount and type of improvement required (Table 14-2 should be used in this selection process).</td>
</tr>
<tr>
<td>6</td>
<td>Preliminary design based on each appropriate ground improvement method</td>
</tr>
<tr>
<td>7</td>
<td>Comparison and selection – final selection is based on performance, constructability, cost, and any other relevant project factors.</td>
</tr>
</tbody>
</table>

Table 14-1 Ground Improvement Design Process
(modified from Elias et al, 2006)
CHAPTER 14
Ground Improvement Technology

<table>
<thead>
<tr>
<th>Strategy</th>
<th>Improved Function</th>
<th>Method</th>
</tr>
</thead>
<tbody>
<tr>
<td>Consolidation</td>
<td>Accelerate primary settlement and/or shear strength and bearing capacity increase, or control of non-uniform or excessive deformations</td>
<td>1. Vertical Drains 2. Temporary Surcharge</td>
</tr>
<tr>
<td>Load Reduction</td>
<td>Reduce settlement, and control non-uniform or excessive deformations</td>
<td>1. Lightweight Fills</td>
</tr>
<tr>
<td>Densification</td>
<td>Increase bearing capacity, and shear strength of granular soils. Decrease settlement and increase resistance to liquefaction and lateral movement, or control non-uniform or excessive deformations</td>
<td>1. Vibro-Compaction 2. Dynamic Compaction 3. Temporary Surcharge</td>
</tr>
<tr>
<td>Reinforcement</td>
<td>Increase shear strength, resistance to liquefaction and lateral movement, and decrease settlement</td>
<td>1. Stone Columns</td>
</tr>
<tr>
<td>Deep Soil Mixing</td>
<td>Increase bearing capacity and shear strength; decrease settlement and/or provide lateral stability and seepage control and resistance to liquefaction and lateral movement</td>
<td>1. Wet Mixing Methods 2. Dry Mixing Methods</td>
</tr>
<tr>
<td>Load Transfer</td>
<td>Decrease settlement, improve resistance to dynamic loading or lateral movement, control non-uniform or excessive deformations</td>
<td>1. Column Supported Embankment (CSE)</td>
</tr>
</tbody>
</table>

Table 14-2 Ground Improvement Categories, Functions and Methods
(modified from Elias et al., 2006)

As indicated in Step 7, the cost of the ground improvement method must be considered in the selection process. Contact the Geotechnical Engineering Bureau for cost information for ground improvements methods previously used by NYSDOT.

According to Ground Improvement Methods, “The success of any ground improvement method is predicated on the implementation of a QA/QC program to verify that the desired foundation improvement level has been reached. These programs incorporate a combination of construction observations, in-situ testing and laboratory testing to evaluate the treated soil in the field”. QA/QC procedures commonly applied are described for each ground improvement method addressed in this chapter.
14.2 PREFABRICATED VERTICAL DRAINS AND OTHER VERTICAL DRAINS

Since the 1940’s, several types of vertical drains have been utilized in diverse ground improvement applications. Those which have been recently used entail sand drains, prefabricated vertical drains, and earthquake drains.

14.2.1 Use of Prefabricated Vertical Drains

Prefabricated vertical drains (PVDs), also commonly called wick drains, are used to accelerate the consolidation and strength gain of compressible soils with low or restricted permeability. 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 into the vertical drains from a compressible soil layer experiencing a temporary water pressure gradient induced by placement of permanent fill and/or a temporary surcharge fill (see Figure 14-1).

PVDs can be viewed as artificial, localized, high permeability drainage paths. The use of multiple PVDs significantly shortens the drainage path (i.e. the distance to a boundary of a faster draining material) for pore water being discharged from a soil layer placed under a new load. Figure 14-2 illustrates a typical prefabricated vertical drain installation for a highway embankment.
The most commonly used PVDs in the U.S. are band-shaped (rectangular cross section) consisting of a synthetic geotextile “jacket” surrounding a plastic core. The jackets are commonly made of commercially available non-woven polyester or polypropylene geotextiles.

Most band-shaped drains are manufactured to dimensions approximately ⅛ inch thick by 4 inches wide. Variations in these dimensions occur in some drains.

To accelerate the rate of settlement, PVDs, are typically installed on a regular grid pattern, either triangular or rectangular, to reduce the flow distance for dissipation of excess pore water pressures associated with the placement of fill. Stone columns discussed later in this Chapter also can provide vertical drainage and similar methods can be applied to evaluate their effect on settlement rates.

14.2.2 Advantages and Disadvantages

The installation of a vertical drain system allows the following to be achieved:

- It decreases the overall time required for completion of primary consolidation.
- It decreases the amount of surcharge load required to achieve the same desired amount of consolidation over a given period of time,
- It increases the rate of strength gain due to consolidation of soft soils when short-term stability is of concern.

![Figure 14-2 Typical Vertical Drain Installation](image)

In 1951, New York State started to use vertical drains consisting of cylindrical columns of free draining sand. Since 1982 only prefabricated vertical drains (PVDs) have been used to accelerate consolidation of compressible soils on NYSDOT projects due to several advantages over the sand drain columns.
Advantages of PV drains versus sand drains:
- Fewer environmental problems concerning disposal of spoil materials.
- Eliminates high cost of sand backfill of drains and material quality control problems.
- Inspection requirements are reduced due to simplicity of installation procedures.
- There is greater assurance of a continuous vertical drainage path (no collapsed holes).
- PV drains can withstand considerable lateral displacement or buckling under vertical or horizontal soil movements.
- Faster rate of installation.

Additional advantages are presented in Ground Improvement Methods.

Disadvantages of PV drains versus sand drains:
- Greater number of wick drains required to achieve same rate of drainage
- Do not provide short term improvement of target soil’s compressive strength
- Headroom limitations (typical equipment is 10 ft taller than wick drain depth)
- Wick drains must be protected from sunlight and large tears. Wick drains should not be used for long term artesian flows
- Equipment mobilization costs may be very high
- Wick drains over 60 foot deep require very tall or specialized equipment
- Sand drains can be designed to accommodate long term artesian conditions and therefore function as pressure relief wells

14.2.3 Feasibility of Prefabricated Vertical Drain Use

The site conditions pertaining to the soil layer to be treated must be evaluated to determine the feasibility for using PVD’s. The following factors, relating to the target soil layer or ground surface, are favorable for their use:

- Moderate to high compressibility potential
- Low or restricted permeability
- The time to achieve at least 85% of primary settlement, without use of vertical drains, will result in excessive construction delays
- Full saturation of target soil layer
- Final embankment or temporary surcharge load increase, at the depth of the target soil layer, must exceed the maximum preconsolidation stress ($\sigma'_p$ or $p'_{c}$)
- Secondary compression will not be significant
- Initial Low-to-moderate shear strength
- Soils normally to slightly overconsolidated (OCR < 1.5)
- Very minimal obstructions (cobbles, boulders, etc) within or above target soil layer
- Site allows relatively flat working surface with at least a 25 foot bench width
- Long term artesian conditions are not likely
- Soil layer to be treated is not deeper than 150 feet
14.2.4 Design of Prefabricated Vertical Drain System

The primary function of PVDs is to reduce the length of the drainage path, thereby decreasing the time for settlement and strength gain to occur within the critical soil layers. Thus, the following procedure can be followed to ensure adequate installation and performance of the PVD system:

1. Note the general surface conditions and perform subsurface investigations at the project site. This may require use of test borings with standard penetration tests and undisturbed samples, cone penetrometers, field vane tests, or flat plate dilatometers. Determine if PVD’s can be utilized to improve unacceptable soil properties.
2. Perform testing to obtain vertical and horizontal Consolidation Coefficient \( (C_v, C_h) \).
3. Predict amount and rate of settlement during and after construction, with and without use of PVD’s.
4. Determine amount of pre-construction settlement and/or stability improvement for adequate performance of the structure to be constructed. If ground stability may become questionable during construction, then staging of new loads can be allowed.
5. Provide the design of a PVD system that incorporates the type, spacing, and depth of the PV drains needed to achieve the soil’s performance requirements within a specified time.

The assumptions used in developing one-dimensional consolidation theory (i.e. vertical drainage of soil medium vs. rate of settlement) were applied to the development of a radial (horizontal) drainage theory which can estimate the rate of settlement with use of a vertical drain column or wick.

The mathematical relationship between the following,

a. time to achieve a desired level of settlement (usually 3 to 5 months),

b. average degree of desired consolidation,

c. assumed diameter of zone drained at each vertical drain,

d. spacing between drains,

e. the soil’s horizontal coefficient of consolidation,

f. soil disturbance during PVD installation

can be shown as:

**Equation 14-1**

\[
t = \frac{D^2}{8C_h}(F(n) + F_s) \ln\left(\frac{1}{1 - U_h}\right)
\]

or

\[
t = \frac{D^2 T_R}{C_h}
\]
where:

\[ t = \text{time required to achieve desired average degree of consolidation} \]
\[ \bar{U}_h = \text{average degree of consolidation to be achieved by PVD system} \]
\[ D = \text{diameter of cylinder of influence of the drain (drain influence zone)} \]
\[ C_h = \text{consolidation coefficient for horizontal drainage} \]
\[ F(n) = \text{drain spacing factor (see equation 14-2)} \]
\[ d = \text{equivalent circular drain diameter} \]
\[ F_s = \text{factor for soil disturbance} \]
\[ T_R = \text{time factor for radial flow (see equation 14-3)} \]

This equation does not account for any consolidation due to vertical drainage through the soil medium. The predicted settlement amounts and rates are based on vertical drainage only through the sand drain columns or wicks.

**Equation 14-2**

\[ F(n) = \ln\left(\frac{D}{d}\right) - 0.75 \]

The following section contains a discussion of each of these components.

### 14.2.5 Determination of Coefficients and Factors

#### Determination of \( F_s \)

Soil disturbance caused by PVD installation is typically ignored except for highly plastic (PI > 21), or sensitive (S_t > 5) soils which can be significantly disturbed during installation of the drains. 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 which keep the drain in position during installation.

#### Determination of \( C_h \)

A reasonable horizontal Consolidation Coefficient (\( C_h \)) can be estimated through vertical consolidation testing of undisturbed soil samples. Yet, even with high quality samples and testing, the laboratory value of \( C_h \) can be off by up to 50 percent from the actual field values. Normally \( C_h \) is greater than \( C_v \) (vertical consolidation coefficient). A conservative approach is to set \( C_h \) equal to \( C_v \). However, for many PVD designs, \( C_h \) can be taken as 1.2 to 1.5 \( C_v \), if no or only slight silt or sand layering is evident in partially dried clay samples. If layering of silt and sand in discontinuous lenses is evident, \( C_h \) may be taken as 2 to 4 \( C_v \).

Determining the horizontal Consolidation Coefficient (\( C_h \)) from field measurements of excess pore water pressures that develop after a surcharge load is applied provides the most accurate \( C_h \) value. This method initially requires the installation of one or more piezometers and the use of test borings to determine the thickness and depth of the compressible soil layer and any.
permeable soil layers. After a uniform surcharge load is placed at the ground surface, changes in the excess pore water pressures are measured at given time periods. This information is then applied to Consolidation versus Depth graphs to obtain a very reliable value for the horizontal Consolidation Coefficient ($C_h$). Although this approach provides a very accurate value, it can often be impractical to use due to space, time, or cost impacts.

A slightly less reliable horizontal Consolidation Coefficient may be assessed in the field by using CPT instrumentation which performs measurements of pore pressure dissipation. Subsurface explorations with CPT equipment can also be used to quickly identify if a thick compressible layer should be evaluated as separate layers with varying soil properties.

**Determination of d**

The equivalent circular drain diameter ($d$) of a PVD can be estimated 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. The diameter chosen generally has little impact on the PVD system’s performance.

**Determination of $\bar{U}_h$**

The average degree of consolidation ($\bar{U}_h$) to be achieved by the PVD system is usually set between 85% to 95% of the total primary consolidation. The chosen degree of consolidation is determined by the amount of post construction settlement that the project or structure can allow. In compressible soil layers less than 20 feet thick, the designer should keep in mind that vertical consolidation by drainage through the soil medium alone can also contribute significantly to the total amount of vertical settlement. This additional settlement, which is not accounted for in the PVD analysis, should be considered when choosing the degree of consolidation required of the PVD design.

Using equation 14-2 with $d =$ diameter of equivalent circular drain (2 in. in NYSDOT design practice), the following factor may be found:

**Equation 14-3**

$$T_R = \frac{F(n) \ln\left(\frac{1}{1-\bar{U}_h}\right)}{8}$$

where:

- $T_R$ = time factor for radial flow
Figure 14-3 Relationship of Drain Spacing (S) to Drain Influence Zone (D) (Rixner et al., 1986)

Table 14-3 $T_R$ for Square Pattern Vertical Drains

<table>
<thead>
<tr>
<th>$\bar{U}_h$ (%)</th>
<th>4 ft.</th>
<th>5 ft.</th>
<th>6 ft.</th>
<th>7 ft.</th>
<th>8 ft.</th>
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<td>10</td>
<td>0.034</td>
<td>0.046</td>
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<tr>
<td>95</td>
<td>0.955</td>
<td>1.311</td>
<td>1.381</td>
<td>1.439</td>
<td>1.490</td>
</tr>
<tr>
<td>99</td>
<td>1.468</td>
<td>2.015</td>
<td>2.122</td>
<td>2.213</td>
<td>2.291</td>
</tr>
</tbody>
</table>
Table 14-4 $T_R$ for Triangle Pattern Vertical Drains

### Determination of $D$

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 ½ feet in coarser soils.

### Determination of $t$

The time ($t$) is the duration required to achieve the desired average degree of consolidation ($\bar{U}_h$) for a chosen diameter of drain influence ($D$) and drain diameter ($d$).

There are three basic variables that can be manipulated in order to achieve a desired result from Equation 14-1. These variables are time, PVD spacing, and surcharge. In order to increase the PVD spacing and reduce the number of PVDs installed, the surcharge can be increased to provide the same amount of consolidation over the same time period. The addition of surcharge and keeping the PVD spacing the same has the effect of reducing the time for consolidation to occur. Typically, time is used as a constant (normally set to meet a specific construction schedule) and the amount of surcharge and the PVD spacing are used as variables.

### Computer Software

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

The water seeping from the top end of the drains is typically discharged out at the base of the embankment or surcharge load. In most cases, this is accomplished using a drainage blanket constructed between the original ground surface and the fill. Site conditions must also allow the continuous flow of water away from the surface load area. If the surficial subgrade material is granular and permeable, a drainage blanket may be of little or no benefit. However, elimination of the drainage blanket should be considered very carefully because its absence may have a severe impact on the efficiency of the drain system.

14.2.7 Instrumentation

All PV drain installations are to be instrumented in order to monitor the progress of settlement and the rate of excess pore pressure dissipation.

Generally, it is best to install the instrumentation after the surface drainage layer and PVD’s have been placed. This allows for an unimpeded site during PVD installation and the ability to accurately locate instrumentation which must be placed close to PVD positions. Additional instrumentation should be installed outside the PVD system’s zone of influence to function as a benchmark.

Within the zone of influence of the PVD system, settlement should be monitored at several locations and depths (i.e. at and below the buried ground surface and within the fill material) in order to determine the rate and amount of consolidation occurring below and within the fill. Piezometers should be installed at multiple levels below the surcharge fill. Changes in the pore water pressure will help identify any detrimental levels of pore pressure buildup and can help in estimating the rate of consolidation. Slope inclinometers may be necessary in locations where slope or lateral stability is critical. If PVD installation may cause ground instability, slope indicators can be installed prior to PVD installation. It is important that both the designer and the personnel responsible for installing the instrumentation have a full understanding of what strata, depth, and location each piece of instrumentation must monitor.
Figure 14-4 Vertical Drains Time for Consolidation
14.2.8 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 14-5. The Contractor is required to submit an installation plan, shop drawings, material samples, and anchorage details. A minimum 12-inch thick layer of clean sand is necessary at the top of the PVDs to provide a drainage path for release of the excess pore pressures. In some applications it will be appropriate to install strip drains across the ground surface to provide horizontal drainage at the top of the PVDs. The drainage layer can be installed as a part of the working platform necessary to make the site accessible to PVD installation equipment.
Figure 14-5 Crane Mounted Installation Rig
(Elias et al., 2006)
When evaluating the contractor’s installation plan or the effectiveness of the PVD system, consideration should be given to other factors including the following:

a. The practical minimum drain spacing is about 3 ft. center to center. Increased soil disturbance effects may eliminate any theoretical benefit of closer spacing.

b. Drain length should be sufficient to consolidate the deposit or portions of the deposit to the extent necessary to achieve the design objectives.

c. The cross-sectional area of the mandrel affects the volume of soil displaced by the mandrel during installation. The amount of soil displacement is intuitively a major factor in the resulting effects of soil disturbance. Typically the cross-sectional area of the mandrel should be less than 10 in².

d. Drain installation disturbs the soil and may reduce the shear strength of the deposit. Where reduced stability is a concern, effects of disturbance on overall stability should be evaluated.

e. Drain layout is typically a triangular or square pattern, with center to center spacings of 3 to 6 ft.

g. To determine the best estimates of $c_h$ and $k_h$ perform "block" permeability tests.

In addition to drain spacing and length, determine the required areal limits of the PV drains. The drains should penetrate any compressible soils where accelerated consolidation is necessary to accomplish the design objectives. Depending on the purpose of the desired consolidation (e.g., reduced post construction settlement or increased stability due to shear strength gain), the areal limits of the drains may extend beyond the plan area of the embankment or other structure.

### 14.2.9 Earthquake Drains

Earthquake (EQ) drains are a subset of PVDs that are used to mitigate/limit the effects of seismically induced liquefaction. While PVDs are thin plastic strips consisting of a rigid core sheathed in filter fabric; EQ drains are perforated, corrugated plastic pipe placed in a filter fabric sock. Earthquake drains can range in size from 1 ½ to 10 inches in diameter, but are more typically 4 to 6 inches in diameter. Earthquake drains are used to reduce the excess pore pressures generated by a seismic event that can lead to liquefaction in loose granular soils (see NYSDOT GDM Chapter 9 for a discussion of liquefaction). The theoretical background for earthquake drains is presented in *FEQDrain: A Finite Element Computer Program for the Analysis of the Earthquake Generation and Dissipation of Pore Water Pressure in Layered Sand Deposits with Vertical Drains*.

EQ drains work by reducing the pore pressure ratio ($r_u$, see equation 14-4), 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 unfavorable levels of fine content (i.e., percent passing the No. 200 greater than 5 percent), and the effect of high accelerations caused by
earthquakes, the \( r_u \) shall be limited to 0.50. Using an \( r_u \) of this magnitude will cause the drain spacing to become smaller and potentially increasing the drain size.

Equation 14-4

\[
r_u = \frac{\Delta u}{\sigma_v'}
\]

where:
- \( r_u \) = pore pressure ratio
- \( \Delta u \) = change in pore pressure
- \( \sigma_v' \) = effective overburden pressure

14.3 VIBRO-COMPACtion

Vibro-compaction is a ground improvement method that uses a specialized vibrating probe for in-situ subsurface compaction of loose sandy or gravelly soils at depths beyond which surface compaction efforts are effective (see Figure 14-6). The vibrating probe densifies loose granular, cohesionless soils by using mechanical vibrations and, in some applications, water saturation to minimize the effective stresses between the soil grains which then allows the soil grains to rearrange under the action of gravity into a denser state.

Generally, vibro-compaction can be used to achieve the following enhanced soil performance or properties:

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

The vibrator is hung from a crane cable or, in some instances; it is mounted to leads in a similar fashion as foundation drilling equipment. The vibrator penetrates under its self weight (or crowd of the machine if mounted in leads) and, at times, with assistance from the action of water jets. The goal is that the vibration and water imparted to the soils transforms the loose soils to a more dense state.
14.3.1 Advantages and Disadvantages and Limitations

14.3.1.1 Advantages

The advantages of this ground improvement technology, as described in *Ground Improvement Methods*, are as follows:

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

14.3.1.2 Disadvantages and Limitations

Vibro-compaction is effective only in granular, cohesionless soils. The realignment of the sand grains and, therefore, proper densification generally cannot be achieved when the granular soil contains more than 12 to 15 percent silt or more than 2 percent clay. The maximum depth of treatment is typically limited to 165 feet, but there are very few construction projects that will require densification to a greater depth.
Like all ground improvement techniques, a thorough soils investigation program is required. Yet, a more detailed soils analysis may be required for vibro-compaction than for a deep foundation design because the vibro-compaction process utilizes the permeability and properties of the in-situ soil to the full depth of treatment to achieve the end result. A comprehensive understanding of the total soil profile is therefore necessary which typically requires continuous sampling or in-situ testing.

Equipment access over the site must also be considered. Since the operation requires use of a large crane, a relatively flat work bench with a width of at least 25 ft must be possible near all areas to be treated.

Wet vibro-compaction requires the use of water to jet the vibrator into the ground. The effluent from the jetting process requires at least temporary containment to allow any fine soil particles to settle out and be disposed. Further, this method of ground improvement may not be acceptable if the existing subsurface environment, either soil or water is contaminated. If contamination is present, use of water jetting may cause its dispersion and therefore other ground improvement methods should be considered.

### 14.3.2 Design and Analysis

The design of vibro-compaction is based mainly on:

1. Analysis of the grain-size distribution of the subsurface soils,
2. The relative density of the soils to be treated, and
3. The level of density improvement required.

As a result of the soil compaction, the operation creates a conical depression at the ground surface and a minor void around the probe, which must also be addressed in the design of the operation. Appropriate soil conditions are shown in Figure 14-7 and backfill materials for the surface depression are as follows.

Use of Figure 14-7, vibro-compaction design vs. grain size distribution:

1. Soils on the coarse side of Zone B may be readily compacted and backfill the operation with sand or gravel,
2. Soils on the coarse side of Zone C may be compacted, but it is advisable to backfill the operation with gravel only,
3. Soils located partially or completely in Zone D are not suitable for vibro-compaction; however soils in Zone D are suitable for vibro-replacement (i.e. stone columns)
As indicated previously, the vibrations induced by vibro-compaction cause the inter-granular forces acting between soil grains to reduce to zero allowing the soil particles to shift under the action of the vibrations and gravity into a more dense state. This more dense state has a reduced void ratio and correspondingly has a reduced compressibility and increase in the shearing resistance of the soil. The achievable reduction in void ratio depends on grain shape, soil composition (gradation), and the probe’s vibration intensity. By controlling the advancement and withdrawal of the vibrator, a compact soil cylinder is formed. The diameter of the cylinder is based on the grain-size distribution, the initial soil density, and the vibrator characteristics. Typical vibrators have dynamic forces that range from 33,750 to 101,250 pounds with frequencies ranging from 1,800 to 2,300 revolutions per minute (rpm). For vibro-compactors operating at lower frequencies, better densification is usually produced. This is because low frequency vibrators allow a greater level of probe movement which translates into a greater
compactive effort. Additionally, the natural frequency of most granular soils is closer to 1,500 rpm than to 3,000 rpm.

The increase in density of the granular soils causes a downward movement of the soil around the vibrator. This downward movement creates a conical depression at the ground surface and a small void around the probe. This depression requires constant filling with additional granular materials. A suitability number ($S_N$) is used to determine if a specific mixture of granular material as backfill material in the vibro-compaction operation is suitable.

The $S_N$ is based on the settling rate of the backfill in water and experience. The $S_N$ is determined using the following equation and a rating criteria presented in Table 14-5 is used to determine the suitability of the specific backfill. The backfill materials consist of sand or sand and gravel, with less than 10 percent by weight passing the #200 sieve and contain no clay.

**Equation 14-5**

$$S_N = 1.7 \left( \frac{3}{(D_{50})^2} + \frac{1}{(D_{20})^2} + \frac{1}{(D_{10})^2} \right)$$

where:

- $D_{50} = \text{grain size diameters for 50% passing in millimeters}$
- $D_{20} = \text{grain size diameters for 20% passing in millimeters}$
- $D_{10} = \text{grain size diameters for 10% passing in millimeters}$

<table>
<thead>
<tr>
<th>$S_N$</th>
<th>0 – 10</th>
<th>11 – 20</th>
<th>21 – 30</th>
<th>31 – 40</th>
<th>&gt; 40</th>
</tr>
</thead>
<tbody>
<tr>
<td>Rating</td>
<td>Excellent</td>
<td>Good</td>
<td>Fair</td>
<td>Poor</td>
<td>Unsuitable</td>
</tr>
</tbody>
</table>

**Table 14-5 Backfill Evaluation Criteria**
*(modified from Elias et al., 2006)*

### 14.3.2.1 Preliminary Design

Determining the increase in dry density of an in-situ soil does not correlate to the amount of improved performance as directly as the increase in the soil’s relative density does. Therefore, the change in relative density is typically the criteria used for evaluating the performance of a vibro-compaction operation. Relative density is an expression that identifies a soil’s level (i.e. percentage) of compaction within the range between its most loose state (0%) to its most dense state (100%). An in-situ soil’s relative density is obtained through the following equation.
Equation 14-6

\[ D_r = \frac{\gamma_n - \gamma_l}{\gamma_d - \gamma_l} \times \frac{\gamma_d}{\gamma_n} \times 100\% \]

where:
- \( \gamma_n \) = dry density of the soil in-situ
- \( \gamma_l \) = dry density of the soil in its loosest state
- \( \gamma_d \) = dry density of the soil in its densest state

Note: The extreme density levels (low and high) are determined from lab tests on soil samples.

Table 14-6 provides the relationship between \( D_r \) and various field tests. Higher \( D_r \) equates to an increase in bearing capacity and a corresponding reduction in settlement. The resistance to liquefaction increases with increasing \( D_r \) and the active earth pressure on an earth retaining structure decreases while the passive earth pressure on an earth retaining structure increases. According to Ground Improvement Methods, “With vibro-compaction, the angle of internal friction is increased on average 5 to 10 degrees, resulting in much higher shear resistance.”

<table>
<thead>
<tr>
<th>Field Test</th>
<th>Very Loose</th>
<th>Loose</th>
<th>Medium Dense</th>
<th>Dense</th>
<th>Very Dense</th>
</tr>
</thead>
<tbody>
<tr>
<td>SPT N-values blows per foot</td>
<td>&lt; 4</td>
<td>5 – 10</td>
<td>11 – 30</td>
<td>31 – 50</td>
<td>&gt; 50</td>
</tr>
<tr>
<td>CPT Tip Resistance tsf</td>
<td>&lt; 51</td>
<td>51 – 102</td>
<td>102 – 154</td>
<td>154 – 205</td>
<td>&gt; 205</td>
</tr>
<tr>
<td>( D_r % )</td>
<td>&lt; 15</td>
<td>16 – 35</td>
<td>36 – 65</td>
<td>66 – 85</td>
<td>86 – 100</td>
</tr>
<tr>
<td>Dry Unit Weight pcf</td>
<td>&lt; 89</td>
<td>89 – 102</td>
<td>102 – 115</td>
<td>115 – 127</td>
<td>&gt; 127</td>
</tr>
<tr>
<td>CSR (^1)</td>
<td>&lt; 0.04</td>
<td>0.04 – 0.12</td>
<td>0.12 – 0.33</td>
<td>0.33 – 0.40</td>
<td>-</td>
</tr>
<tr>
<td>Shear Wave Velocity, ( V_s ) fps</td>
<td>&lt; 394</td>
<td>395 – 525</td>
<td>526 – 656</td>
<td>657 – 738</td>
<td>&gt; 739</td>
</tr>
</tbody>
</table>

CSR \(^1\) - Cyclic Stress Ratio Causing Liquefaction

Table 14-6 Apparent Density Levels vs. Field Test Results (modified from Elias et al., 2006)

The degree of \( D_r \) improvement is not only affected by the gradation of the soil, but also by the area influenced around the compaction point. Figure 14-8 provides an example of an approximate relationship between \( D_r \), soil type and treatment area for a specific vibrator. The
increase in $D_r$ is limited to 85 percent since, at this density, the improvement to the soil is enough to effectively increase bearing capacity and resistance to liquefaction and reduce settlement.

\[90\]

\[70\]

\[50\]

\[2.0\]

\[5.5\]

\[9.5\]

\[13\]

\[\text{Relative Density (Percent)}\]

\[\text{Area Per Compaction Point (m}^2\text{)}\]

| Silty Sand (5% - 15% Sil) | Uniform, Fine to Medium Sand (clean) | Well-Graded, Clean Sand |

**Figure 14-8 Variation of $D_r$ with Tributary Area**

(Elias et al., 2006)

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

A typical vibro-compaction program is designed with various probe spacing and patterns. The distance between compaction points is critical, as the density improvement generally decreases as the distance from the probe increases. Stronger vibroprobes allow for wider spacing under the appropriate soil conditions.
The compaction point pattern affects the progression of densification over large areas. An equilateral triangular pattern is primarily used to compact large areas, since it is the most efficient pattern. The use of a square pattern instead of an equilateral triangular pattern requires 5 to 8 percent more points to achieve the same minimum densities over large areas.

Given the in-situ soil gradation and relative density required, the spacing of compaction points can be determined. Figures 14-9 and 14-10 show typical area patterns and spacing for 80 percent relative density requirements. The spacing of the vibro-compaction points would be wider for lower relative density requirement. However, for most vibro-compaction projects, these patterns are appropriate as they meet the performance criteria of 70 – 75 percent relative density for column or bridge footings and 80 percent relative density for mat foundations.

**Figure 14-9 Typical Compaction Point Spacing for Area Layouts**

(Elias et al., 2006)
Figure 14-10 Typical Compaction Point Layouts for Column Footings
(Elias et al., 2006)
14.4 STONE COLUMNS

Stone columns are constructed using down-hole vibratory probe methods similar to those used in vibro-compaction (See previous Section). The main difference is that instead of using coarse-grained soil to simply fill the void created by the vibro-compaction operation, stone or other clean, coarse grained materials are placed, and compacted, to form a narrow structural element (i.e. a column) which functions as one or more of the following:

1. enhance the average shear strength and bearing capacity of a weak soil mass,
2. transfer a surface load to deeper competent materials, or
3. provide easy drainage of temporarily high pore water pressures.

This Section will discuss the design considerations and use of stone columns and describe variations of technique, such as vibro-concrete columns (VCCs), geotextile-encased columns (GECs), and Geopier® Rammed Aggregate Pier™ (Geopiers).

Stone columns are ideally suited for improving soft silts and clays and loose silty sands. Stone columns under suitable conditions will:

- increase a soil’s bearing capacity and shear resistance
- reduce settlements,
- increase the time-rate of consolidation,
- reduce liquefaction potential, and
- stabilize existing slopes affected by low shear strength soils.

Stone columns, in general, are most economically attractive for sites requiring column lengths less than 35 ft. deep and preferably about 20 ft. deep below the surface.

Unsuitable soil conditions for stone columns include soils having thick layers of very soft or sensitive clays and organic materials. If the thickness of the unsuitable soil layer is more than the diameter of the stone column, then stone columns may not be appropriate because the very soft soils will not provide adequate lateral support of the stone column. In addition, stone column construction can be hampered by the presence of a thick, dense overburden, or soils with boulders, cobbles or other obstructions that may require pre-drilling prior to installation of the stone column.

As described in Table 14-7, Stone columns are constructed using either a vibro-replacement or vibro-displacement installation with the stone aggregate placed using either top or bottom feed methods (see Figures 14-11 and 14-12, respectively).
### Table 14-7 Vibro-replacement and Vibro-displacement Definitions (Elias et al., 2006)

<table>
<thead>
<tr>
<th>Method</th>
<th>Definition</th>
</tr>
</thead>
<tbody>
<tr>
<td>Vibro-replacement</td>
<td>Refers to a wet installation method in which water jetting is used to flush soil out an uncased hole to aid ground penetration by the vibrator. After the hole is created, the removed soil is then replaced by aggregate backfill material top-fed from the surface and compacted, in lifts, at the base of the probe.</td>
</tr>
<tr>
<td>Vibro-displacement</td>
<td>Refers to a dry installation method where almost no in-situ soil is removed. Rather, the soft soil is displaced by the probe and the aggregate backfill material (fed at the top or bottom of the hole), pressed and compacted into the soft soils, to form an aggregate column.</td>
</tr>
</tbody>
</table>

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 14-11.

![Figure 14-11 Applicable Grain-Size Distributions for Stone Columns (Elias et al., 2006)](image-url)
**Vibro-Replacement**

Vibro-replacement involves a wet installation method that replaces deep, narrow pockets of the in-situ soil with stone aggregate columns. In this method a high-pressure water jet, located at the tip of the probe, is used to excavate a narrow, open (uncased) hole. Once the hole is progressed to the design depth, the hole is flushed out several times by raising and dropping the probe to remove any loose silt and sand at the bottom of the hole. The vibro-probe is retracted and a limited amount of stone is placed into the hole from the top. The probe’s vibration mode is turned on and it is inserted into the hole to compact the lift of stone. The probe is retracted again and the process repeated until the stone column is formed to the ground surface. During the entire operation, water is continually pumped into the hole to prevent collapse and to keep the aggregate clean. This method is best suited for sites with soft to firm soils with undrained shear strengths of 200 to 1,000 psf and a shallow groundwater table, and where drill wash and spoil containment and disposal can be practically handled.

**Vibro-Displacement**

When a cleaner or lesser environmental impact operation is preferred, stone columns should be constructed using the vibro-displacement method. The operation is a dry installation method where the stone aggregate can be placed into the hole from the top or from aggregate ports at the bottom of the probe. Although the probe’s dead weight and vibration, in lieu of water jetting, is used to excavate the hole, air jetting and/or pre-augering may be used to prevent clogging of the aggregate ports or to assist in advancing or extracting the probe. This method is best suited for sites where collapse of the hole during the column’s installation is unlikely.

![Figure 14-12 Top Feed Construction Method (Elias et al., 2006)](image-url)
Figure 14-13 Bottom Feed Construction Method
(Elias et al., 2006)
The stone backfill is compacted by the use of a vibrating probe which typically varies in diameter from 12 to 18 in. To construct the column, the hole is excavated and then backfilled with a clean, coarse aggregate in 1 to 4 ft. lifts. Stone is either: dumped from the ground surface and allowed to fall through the annular space provided between the probe and the sides of the enlarged hole, or the stone is placed in a hopper where the stone exits from ports at the base of the probe. Each lift is penetrated several times with the vibrating probe to compact the stone and force it into the surrounding soil. Successive lifts are placed and densified until a column of stone has been formed up to the ground surface.

Since stone columns backfilled stone aggregate derive much of their lateral strength and settlement characteristics from the surrounding soil, they do not perform well in very soft clay (i.e. shear strength < 150 psf) or in soils containing peat layers with a thickness greater than the diameter of the column. Variations to the standard stone column design have been created in order to allow use of this system in problematic soils and are discussed in the following paragraphs.

**Vibro-Concrete Columns**

Vibro-concrete columns (VCCs) were developed to treat soils with the above constraints. 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 expected of the vibro-systems, with the load carrying characteristics of a deep foundation.

The vibro-concrete column process employs a bottom feed vibrator that can penetrate the soils to a level suitable for bearing. Concrete is pumped through the vibrator assembly during initial withdrawal. The vibrator then repenetrates the concrete, displacing it into the surrounding soil to form a high-capacity, enlarged column base. The vibrator is then slowly withdrawn as concrete is pumped and maintained at a pressure to form a continuous shaft of concrete up to the ground level. At ground level, a slight mushrooming of the concrete column is constructed to assist the transfer of the applied loading into the vibro-concrete column (see Figure 14-14).

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.
Geotextile Encased Columns

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

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

**Geopier® Columns**

Geopier® Rammed Aggregate Pier™ (Geopiers) is a variant of stone columns, but instead of creating the hole with a vibrating probe, 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 14-15). Geopiers typically extend to depths of 6 to 33 feet.

![Figure 14-15 Geopier® Rammed Aggregate Pier™ (Elias et al., 2006)](image)

Geopiers are most applicable to supporting structural foundations placed over relatively shallow, soft to stiff cohesive soils with undrained shear strengths ranging from 300 to 4,000 psf or for foundations placed over shallow, loose to medium dense silty and clayey sands. The soil must be stable as an open hole, or as a partially cased hole, during placement and compaction of the
gravel backfill. The gravel is placed in relatively thin lifts with the first lift of gravel heavily compacted with a hydraulic hammer to form a bulb at the bottom of the pier, thus pre-stressing 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 compacts the gravel and displaces it laterally into the sidewalls of the hole. This action increases the shear strength in the surrounding soil, further stiffening the stabilized composite soil mass.

**14.4.1 Typical Applications for Stone Columns**

Stone columns are typically used to address the following conditions:

1. Stone columns can be used to improve the stability of a slope by creating discrete zones of high shear strength within a low shear strength soil mass and thereby increase the average resistance to movement along any potential failure surface.

2. Stone columns can enhance the performance of a low bearing capacity soil layer by transferring most of a heavy surface load to a deeper, stronger layer. Further bearing capacity improvement can be accomplished by densification of the in-situ soils through the use of vibro-displacement methods.

3. Stone columns can be used to reduce the amounts of total and differential settlement that a new embankment would experience if placed over a low strength soil.

4. Stone columns will provide a conduit for the flow of ground water under excess pore pressure, thus decreasing the time for settlement to occur below a new embankment. Also, the use of stone columns can further decrease the time required for placement of a large fill by allowing construction to proceed immediately instead of waiting for settlement to stabilize after placement of a temporary surcharge.

5. Stone columns are used to mitigate the potential for liquefaction induced by a seismic event through densification of loose, in-situ sandy soils and by providing pore pressure relief zones which have a far greater hydraulic conductivity than the in-situ sands. The installation of stone columns can also improve the cyclic resistance ratio of the soil mass.

When installed under the appropriate site conditions, the advantages of stone columns are lower costs and technical feasibility when compared to the use of deep foundations for the support of embankments or shallow foundations. Stone columns also provide a less expensive option to removal and replacement of shallow, weak soils, particularly on large sites with shallow groundwater. In developed areas where strong vibration methods such as dynamic compaction, deep blasting, or pile driving would have a negative impact on adjacent properties, the weaker vibration installation of stone columns may provide a viable alternative to ground improvement.

**14.4.2 Feasibility Evaluation for the Use of Stone Columns**

Clayey soils, and most silts are not readily improved by the installation vibrations of stone columns, and instead, the improvement in these soils is affected more by the amount of soft soil replaced and displaced by the stone columns, VCC, GEC, or Geopier. Therefore, the feasibility analysis for stone columns must primarily evaluate the stone columns installation and long term function.
A summary of the factors affecting the feasibility of utilizing stone columns at soft ground sites is as follows:

1. The allowable design loading above a stone column group should be relatively uniform and limited to a maximum of 112.5 kips per column, if sufficient lateral support by the in-situ soil can be developed.

2. The most cost effective site improvement is likely to be obtained in compressible silts and clays occurring within 33 feet (10 m) of the surface and generally ranging in shear strength from 300 to 1000 psf. Stone columns installed to greater depths are possible, but problems and delays related to hole collapse and aggregate placement become more likely. Sites which may require a large amount of pre-boring of holes to provide access of the probe to underlying soft soils may become cost prohibitive.

3. Stone columns should not be used in highly sensitive soils which have sensitivity values greater than 5. Special care must be taken when using stone columns in soils containing organics and peat lenses or layers with undrained shear strength less than 200 psf. Because of the high compressibility and low strength of these materials, little lateral support may be developed and large horizontal deflections or settlement of the columns may result. When the thickness of the organic layer is greater than one to two stone column diameters, the ability to develop consistent column diameters becomes questionable.

4. Ground improvement with stone columns reduces settlements typically anticipated of low strength soils by thirty to fifty percent-and reduces the amount of anticipated differential settlement by five to fifteen percent.

5. Stone columns have been used in clays having localized, minimum (i.e. not average) undrained shear strengths as low as 150 psf, but this level of strength should not be viewed as the allowable minimum when considering the use of stone columns. Instead, the average shear strength minimum at a site should be approximately 300 psf, and caution should be exercised in constructing any stone column in soils with shear strengths less than 400 psf due to a high probability of hole collapse and the intrusion of soft soil into the clean aggregate column. At sites with soils having an average shear strength less than 360 psf, the use of clean sand in lieu of clean aggregate can be considered to prevent soft soil intrusion into the column. Due to the development of excessive resistance to penetration of the vibrator and economic considerations, a practical upper limit is in the range of 1,000 to 2,000 psf undrained shear strength.

6. Individual stone columns are typically designed for a bearing load of 20 to 30 tons (40 to 60 kips) per column. When used to support the foundation of a structure, the ultimate capacity of a group of stone columns is predicted by estimating the ultimate capacity of a single column and multiplying that capacity by the number of columns in the group.

7. Stone columns have been used effectively to improve stability of slopes and embankments if any possible failure surface is located less than 30 feet from the ground surface. The slope stability design is usually based on conventional slip circle or wedge analyses utilizing composite (i.e. averaged) shear strengths. A relatively flat work bench with a width of at least 25 feet is needed for operation of the crane on or near the slope. Stone columns can be also be utilized to reduce seepage or artesian forces that encourage slope movement.
8. The following relationship is recommended to prevent piping of the soil into the aggregate of the stone column:

**Equation 14-7**

\[ 20D_{S15} < D_{G15} < 9D_{S85} \]

where:
- \( D_{S15} \) = diameter of the surrounding soil passing 15%
- \( D_{G15} \) = diameter of stone (gravel) passing 15%
- \( D_{S85} \) = diameter of the surrounding soil passing 85%

*Stone Gradation: The gradation selected for design should (1) follow a gradation that can be economically and readily supplied and (2) be coarse enough to settle rapidly in water to the base of the probe.*

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

1. The allowable design load for VCC is a function of the diameter of the column, the allowable strength of the concrete, and the strength of the column’s bearing layer. Typical column diameters range from 18 to 24 inches. Typical allowable design loads for VCC range from 75 to 100 tons.
2. VCC are typically used in very soft clay and organic soils where standard stone columns may not be appropriate.
3. Typical VCC lengths vary from 16 to 33 feet.

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

1. GEC may be installed in soft, compressible clays up to depths of approximately 33 feet. Typical column diameters range from 2 to 3 feet.
2. GEC allowable load capacity is 20 to 40 tons.
3. Settlement of GEC typically occurs during construction of embankment and may experience up to 10 to 20 inches of settlement.

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

1. The allowable design load for a Geopier is typically in the range of 25 to 75 tons per pier, depending on the lateral confinement provided by the surrounding soils (i.e., undrained shear strength ≥ 300 psf for soft saturated clays and SPT N-value ≥ 1 blow per foot for cohesionless soils).
2. Geopiers have been used effectively in soft soils, provided that top-of-pier stresses are lower than those needed to initiate top of pier bulging into the soft soils.
3. The installation of Geopiers in soils that do not stand open during drilling (loose granular soils, very soft cohesive soils) often requires the use of temporary casing, which reduces the installation rate and increases the cost of the piers.
4. The maximum practical depth of Geopiers is limited to 33 feet.

14.4.3 Environmental Considerations

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

14.4.4 Design Considerations

The available design methods of stone columns currently involve an empirical process; however, preliminary design guidelines have been developed and are provided below. Additional information may be obtained from the following references.

1. Design and Construction of Stone Columns, Volume I, FHWA/RD-83/026

Preliminary Design Considerations:
For stone columns to adequately perform in any application, the soils surrounding the columns must provide sufficient lateral support to prevent excessive bulging at any point along the column. To prevent a local shear failure within the stone column, adequate lateral support is most critical between the top of the column to a depth equal to 3 times the column diameter. If very soft soil exists near the ground surface, an uncompacted mat (1 to 3 feet thick) of sandy or gravelly material should be placed at the surface to facilitate construction and to provide overburden pressure that will improve the top of column’s resistance to local shear.

For heavy load support applications, the tip of the columns should terminate in a compact soil layer to prevent bearing failures. In slope stability applications, the tip of the stone column does not need to be set into a good bearing layer unless a large surcharge is being added to enhance the frictional resistance within the stone columns.

Stone columns are typically stiffer than the in-situ materials that surround the columns; therefore, the columns will provide larger portion of the shearing or bearing resistance to the applied load. Although the stone columns may only be a small portion of the foundation area supporting a heavy vertical load, the stone columns carry a greater percentage of the applied vertical load by soil arching between the tops of the stone columns.

In settlement, stability and bearing analyses, a composite shear strength of the soil-stone column matrix is used where each row of the stone column group may be given its own shear strength value. Determining the composite shear strength value depends on the empirical method being used and often involves a complex combination of stress concentrations at the columns, an area
replacement ratio, overburden changes between rows of columns, and a weighted unit weight average for the soil-stone column matrix. The reader is directed to the references listed above detailed guidance for determining the composite shear strength value.

### 14.4.4.1 Unit Cell Concept for Design Analysis

A design analysis should assume that the vertical or shearing loads applied to a stone column reinforced soil mass will not be resisted solely by the stone columns, but rather the resisting force is shared between the stone columns and the in-situ soil surrounding each stone column. But, the resisting force is not shared equally between the two materials. To incorporate this condition into a design analysis, the concept of the “unit cell” was developed.

According to *Ground Improvement Methods*, “For purposes of settlement and stability analyses, it is convenient to associate the tributary area of soil surrounding each stone column with the column, as illustrated in Figure 14-17. Although the tributary area forms a regular hexagon about the stone column, its top cross section can be closely approximated as an equivalent circle having the same diameter. The resulting equivalent cylinder of material having a diameter (De) enclosing the tributary soil and one stone column is known as the “unit cell”. The stone column is centered within the boundary of the unit cell.

For design, the diameter (De) of the “unit cell” column area can be estimated as being 5 to 13 percent greater than the center to center distance between adjacent stone columns. The estimated diameter (De) of the unit cell is based upon the installation pattern for the stone columns.

Triangular Grid pattern of installation:
**Equation 14-8**

\[ D_e = (1.05)(S) \]

Square or Rectangular Grid Pattern of installation:
**Equation 14-9**

\[ D_e = (1.13)(S) \]

where:
- \( D_e \) = diameter of the unit cell column area
- \( S \) = center-to-center spacing between stone columns
Figure 14-16 Stone Column Equilateral Triangular Pattern
(Elias et al., 2006)

Figure 14-17 Unit Cell Idealization
(Elias et al., 2006)
14.4.4.2 Area Replacement Ratio

The amount (percentage) of soft soil actually replaced by the stone aggregate column has a significant effect on the final performance of the improved soft soil layer.

The Area Replacement Ratio \((\alpha_s)\) indicates the area of the soft soil surface that is replaced by the stone aggregate. The ratio’s value also indicates the amount of area at the top of the unit cell column that is taken up by the stone column. The more soft soil replaced by the stone column, the larger the area replacement ratio, and the greater the positive effect on performance of the soil mass.

Typical values of the area replacement ratio \((\alpha_s)\) range from 0.10 to 0.40; but for most applications, the replacement ratio is greater than 0.20. These area replacement ratios can be viewed as indicating that 10 to 40 percent of the weak soil is replaced by stone columns with most applications using a value near the 20% replacement amount.

Equation 14-10

\[
\alpha_s = \frac{A_s}{A}
\]

Equation 14-11

\[
a_s = \frac{1}{\alpha_s} = \frac{A}{A_s}
\]

where:
- \(\alpha_s\) = area replacement ratio
- \(A_s\) = top edge area of the stone column using the initial hole diameter
- \(A\) = top edge area within the unit cell column with diameter \(D_c\)
- \(a_s\) = area improvement ratio

14.4.4.3 Spacing and Diameter of Stone Column Holes

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

“Triangular, square or rectangular grid patterns are used with center-to-center column spacing of 5.0 to 11.5 feet. For structural footing support, the stone columns are installed in rows or clusters. For both structural footing or wide area support, the stone columns should extend beyond the loaded area.” Spacings less than 5 ft. are not recommended for the wet method of installation due to possible excavation disturbance to adjacent, completed holes.

In NYSDOT, designs for stone columns are usually developed using the compact equilateral
triangular pattern instead of a square pattern. Center to center spacings used for stone columns commonly vary from about 6 ft. to 9 ft. with typical values being 7 ft. to 8 ft.

### 14.4.4.4 Stress Ratio

The degree of sharing of the applied load between the stone columns and the in-situ soils depends on the relative stiffness of the stone column to the in-situ soils, and the spacing and diameter of the stone columns. Because the stone columns and the in-situ soils deflect (strain) approximately the same amount, the stiff stone columns must carry a greater portion of the load than the soft in-situ soils. This concept of equal deflection has also been called the equal strain assumption and has been verified by both field measurements and finite element analysis. The ratio between the stress (load/area) carried by the stone column and the stress (load/area) carried by the in-situ soil is defined in the following equation:

**Equation 14-12**

\[
n = \frac{\sigma_s}{\sigma_c}
\]

where:
- \(n\) = stress ratio or stress concentration
- \(\sigma_s\) = stress on the stone column
- \(\sigma_c\) = stress on the surrounding soil

Measured values of “n” have generally been between 2.0 and 5.0 for stone column groups used to support vertical loads. Settlement theory suggests that “n” should increase to a higher value with time. A high n-value (3 to 4) may be assumed if very weak soils are present and the column spacing is tight. Lower values of “n” (2 to 2.5) may be assumed when the surrounding soil is of slight to moderate weakness and the column spacing is wider. For preliminary design, a conservative n-value of 2.5 should be assumed unless more accurate values are developed through experience.

For landslide mitigation, where little to no vertical load is applied at the top of the stone columns, the n-value is equal to 1.0.

For vertical load support applications, equilibrium of the vertical forces shared between the soil and stone columns can be assumed for a given \(\alpha_s\) as portrayed by the following equation.

**Equation 14-13**

\[
q = \sigma_s \alpha_s + \sigma_c (1 - \alpha_s)
\]

where:
- \(q\) = applied vertical stress at the top of the unit cell
Estimates of the separate stresses between the stone column and the surrounding soil in the unit cell can be determined by rearranging the above equation. These estimated stress levels in the stone column and surrounding soil can then be used in a settlement and bearing stability analysis.

**Equation 14-14**

\[
\sigma_s = \frac{q}{[1 + (n-1)\alpha_s]} 
\]

**Equation 14-15**

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

### 14.4.4.5 Final Design

Settlement, stability, or bearing capacity analysis for stone column applications is typically handled through empirical methods which incorporate the stone column spacing and diameter, installation pattern, stress ratio, the area replacement ratio, and variations on the unit cell concept. Detailed guidance and discussion on the various empirical design methods are presented in the references listed in Section 14.4.4. The reader is directed to these references for further design information.

### 14.4.5 Verification

According to *Ground Improvement Methods*, “In-situ testing to evaluate the effect of the stone column construction on the native cohesive soil can be also specified. However, the specified test method should be selected on the basis of its ability to measure changes in lateral pressure (due to increased pore water pressure or soil density) in cohesive soils. The electric cone penetrometer test (CPT), the flat plate dilatometer test (DMT) and the pressuremeter test (PMT) appear to provide the best means for measuring the change, if any, in lateral stress due to stone column construction.”
Figure 14-18 Stone Column Elevation View

Figure 14-19 Stone Column Plan View
14.5 DYNAMIC COMPACCIÓN

Dynamic compaction is the process of ground improvement using weights dropped from a height resulting in the application of high energy levels to the in-situ soil resulting in improvement of the soil. Typically, the weight (called a tamper) ranges from 11 to 39.6 kips and is dropped from heights of 30 to 100 feet. Dynamic compaction can typically be performed using conventional construction equipment as long as the crane has a free spool attached to allow the cable to unwind with minimal friction. The depth of improvement generally ranges from 10 to 36 feet for light- and heavy-energy applications, respectively. The light-energy applications consist of low weights and low drop heights, while heavy-energy applications consist of heavy weights dropped from high heights. Figure 14-21 provides a schematic of dynamic compaction.
14.5.1 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, granular soils (Zone 1 - sands, gravels and non-plastic silts) that meet the gradation, permeability (hydraulic conductivity) and plasticity shown in Figure 14-22. For saturated Zone 1 soils, the induced excess pore pressures from
dynamic compaction cause the soil particles to lose point-to-point contact (i.e. liquefy). Following dissipation of these excess pore pressures, the soil grains settle into a more dense structure. Besides permeability, the degree of saturation, length of the drainage path, and the soil stratigraphy also affect the effectiveness of dynamic compaction. The degree of saturation is related to the position of the groundwater table. For soils located above the groundwater table, the results of dynamic compaction are immediate, while time is required to allow pore pressure dissipation of soils below the water table. Dense or hard layers near the ground surface can limit the effect of dynamic compaction on deeper soils.

Figure 14-22 Soil Grouping for Dynamic Compaction
(Elias et al., 2006)
<table>
<thead>
<tr>
<th>General Soil Type</th>
<th>Degree of Saturation</th>
<th>Suitability for Dynamic Compaction</th>
</tr>
</thead>
<tbody>
<tr>
<td>Pervious deposits in the grain size range of boulders to sand with 0% passing the #200 sieve Coarse portion of</td>
<td>High or Low</td>
<td>Excellent</td>
</tr>
<tr>
<td>Pervious deposits containing not more than 35% silt Fine portion of</td>
<td>High</td>
<td>Good</td>
</tr>
<tr>
<td>Semi-pervious soil deposits, generally silty soils containing some sand but less than 25% clay with PI&lt;8</td>
<td>High</td>
<td>Fair</td>
</tr>
<tr>
<td>Impervious soil deposits, generally clayey soils where PI&gt;8</td>
<td>High</td>
<td>Not recommended</td>
</tr>
<tr>
<td>Miscellaneous fill including paper, organic deposits, metal and wood.</td>
<td>Low</td>
<td>Fair-long term settlement anticipated due to decomposition. Limit use to embankments.</td>
</tr>
<tr>
<td>Highly organic deposits peat-organic silts.</td>
<td>High</td>
<td>Not recommended unless sufficient granular fill added and energy applied to mix granular with organic.</td>
</tr>
</tbody>
</table>

**Table 14-8 Suitability of Deposits for Dynamic Compaction**

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 14-23). It should be noted that while the void ratio decreases, the volume of the solids does not change.
The soils indicated in Zone 3 (Figure 14-22) are typically impervious, plastic, fine-grained soils. The use of dynamic compaction is not recommended for these soils. The soils located in Zone 2 may be improved using dynamic compaction; however, multiple passes of the tamper will be required. In addition, additional time will be required between each pass to allow for the dissipation of excess pore pressures.

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

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

14.5.1.1 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.

14.5.1.2 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 6.5 feet below the existing ground surface to prevent softening of the surface soils and to limit the potential of the tamper sticking in the soft ground.
- A working platform may be required above very loose deposits. The working platform also functions to reduce the penetration of the tamper. The cost of the working platform can add significant costs to the project.
- Large lateral displacements (1 to 3 inches) have been measured at distances of 20 feet from the point of impact by tampers weighing 33 to 66 kips. Any buried structures or utilities within this zone of influence could be damaged or displaced.
14.5.1.3 Environmental Considerations

As indicated previously the vibrations created by dynamic compaction can have an adverse effect on adjoining properties. According to Ground Improvement Methods, “The U.S. Bureau of Mines has found that building damage is related to particle velocity. Figure 14-24 was developed by the Bureau based on experiences with damage measurements made in residential construction from blast-induced vibrations. The limiting particle velocity depends upon the frequency of the wave form. Normally, dynamic compaction results in frequencies of 5 to 12 Hertz (Hz). Using Figure 14-24 as a guide, this would limit peak particle velocities to values of ½-inch per second for older residences with plaster walls and ¾ inches per second for more modern constructions with drywall. Peak particle velocities that exceed the values given in Figure 14-24 do not mean damage will occur. Rather, these values are the lower threshold beyond which cracking of plaster or drywall may occur.

“Data generated by the U.S. Bureau of Mines indicate that minor damage occurs when the particle velocity exceeds 2 inches per second (51 mm/sec), and major damage occurs when the particle velocity exceeds about 7 ½ inches per second (190 ½ mm/sec). Thus, keeping the particle velocity less than about ½ to ¾ inches per second should be a reasonably conservative value to minimize damage.”
Seismographs are typically used to measure ground velocities caused by dynamic compaction. Typically, a base line reading is obtained prior to commencing operations to obtain the level of ambient background vibrations. The readings during production operations are obtained from seismographs on adjacent structures or at the construction limits. However, prior to dynamic compaction production operations, an estimate of the particle velocity to be generated is required. Figure 14-25 can be used for planning purposes.

![Figure 14-25 Scaled Energy Factor vs. Particle Velocity](Elias et al., 2006)

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.

14.5.2 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.

14.5.2.1 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 ground improvement using SPT or CPT methods. The approximate required level of improvement should be determined for the specific baseline testing procedure. Verification testing shall be conducted during the dynamic compaction operations to determine if the required amount of densification is being achieved.

14.5.2.2 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.

**Equation 14-16**

\[ D_{\text{max}} = n \sqrt{(W)(H)(2.9756)} \]

where:
- \( D_{\text{max}} \) = maximum depth of improvement (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 (tons)
- \( H \) = drop height (ft.)

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.
14.5.2.3 Degree of Improvement

As indicated above, the degree of improvement is typically measured using either SPT or CPT measurements. SPT or CPT tests are performed prior to and after dynamic compaction to monitor the amount of improvement imparted on the soil. Figure 14-26 provides a general indication of the amount of improvement from dynamic compaction.
Figure 14-26 Dynamic Compaction Improvements vs. Depth (Elias et al., 2006)

- Initial Stages of Tamping
- Increasing Improvement
- Surface Deposits Loosened to Depth of Crater Penetration
- Initial Condition
- Note:
  
  \[ D = \text{maximum depth of improvement predicted by equation 1.} \]
  
  \[ M = \text{maximum improvement usually occurs around } D/3 \text{ to } D/2. \]

- After Densification Including Ironing Pass
- Maximum Improvement at about \( D/3 \) to \( D/2 \)
The degree of improvement achieved is primarily a function of the average energy applied at the ground surface. Generally, the greater the amount of energy, the greater the degree of improvement; however, there are limitations to the maximum SPT or CPT values that can be achieved. These maximum values are listed in Table 14-9. 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>
<thead>
<tr>
<th>Soil Type</th>
<th>Maximum Test Values</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>N-values (bpf)</td>
</tr>
<tr>
<td>Sand &amp; Gravel</td>
<td>30 – 50</td>
</tr>
<tr>
<td>Sandy Silts</td>
<td>25 – 35</td>
</tr>
<tr>
<td>Silts &amp; Clayey Silts</td>
<td>20 – 35</td>
</tr>
<tr>
<td>Clay fill &amp; Mine Spoil</td>
<td>20 – 40&lt;sup&gt;1&lt;/sup&gt;</td>
</tr>
<tr>
<td>Landfills</td>
<td>15 – 40&lt;sup&gt;1&lt;/sup&gt;</td>
</tr>
</tbody>
</table>

<sup>1</sup> Higher test values may occur because of large particles in the soil mass.

**Table 14-9 Upper Bound Test Values after Dynamic Compaction**

*(Elias et al., 2006)*

### 14.5.2.4 Energy Requirements

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

**Equation 14-17**

$$AE = \frac{(W)(H)(N)(P)}{(G)^2}$$

where:
- $AE$ = applied energy
- $N$ = number of drops at each specific drop point location
- $W$ = tamper weight
- $H$ = drop height
- $P$ = number of passes
- $G$ = grid spacing

The average applied energy is the sum of all different size tampers and drop heights. Normally, high energy is achieved using a heavy tamper dropped from a high height. This is frequently
followed by the ironing pass (low level energy). The ironing pass is conducted using smaller sized tampers being dropped from lower heights. For planning purposes, the estimated required energy can be obtained from Table 14-10.

<table>
<thead>
<tr>
<th>Soil Deposit</th>
<th>Unit Applied Energy (ft-lb/ft²)</th>
<th>Percent Standard Proctor Energy¹</th>
</tr>
</thead>
<tbody>
<tr>
<td>Zone 1 Soils</td>
<td>4130 – 5170</td>
<td>33 – 41</td>
</tr>
<tr>
<td>Zones 2 and 3 Soils²</td>
<td>5170 – 7230</td>
<td>41 – 60</td>
</tr>
<tr>
<td>Landfills</td>
<td>12400 – 22700</td>
<td>100 – 180</td>
</tr>
</tbody>
</table>

¹ Standard Proctor energy equals 12400 ft-lb/ft²
² Refer to Figure 14-22

**Table 14-10 Applied Energy Guidelines**
*(Elias et al., 2006)*

### 14.6 DEEP SOIL MIXING

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

<table>
<thead>
<tr>
<th>Approach</th>
<th>Abbreviation</th>
</tr>
</thead>
<tbody>
<tr>
<td>Method of Reagent Injection</td>
<td>Wet (W) or Dry (D)</td>
</tr>
<tr>
<td>Method of Reagent Mixing</td>
<td>Rotary energy (R) or High-pressure Jet (J)</td>
</tr>
<tr>
<td>Location of Mixing Action</td>
<td>End of Drilling Tool (E) or Along Shaft (S)</td>
</tr>
</tbody>
</table>

**Table 14-11 Deep Soil Mixing Generic Terms**
*(Elias et al., 2006)*

These generic terms can be combined into four distinct processes of deep soil mixing (see Figure 14-27), WRS, WRE, WJE and DRE. Some of the possible combinations of deep soil mixing methods do not exist. For example DJE (dry, jet end) does not exist. Jetting is a wet method and, therefore, could not be used with a dry mix application.
Figure 14-27 Generic Classification of Deep Soil Mixing Techniques
(Elias et al., 2006)
The four processes discussed previously can be divided into two groups as indicated in Table 14-12.

<table>
<thead>
<tr>
<th>Method</th>
<th>Group</th>
<th>Description</th>
</tr>
</thead>
<tbody>
<tr>
<td>Wet Deep Soil Mixing Methods</td>
<td>WRS, WRE, WJS</td>
<td>Refers to wet, single or multi-auger, block or wall developed for large-scale foundation improvement in any soil. Primary reagents are cement-based.</td>
</tr>
<tr>
<td>Dry Deep Soil Mixing Methods</td>
<td>DRE</td>
<td>Refers to dry, single-auger column technique developed for soil stabilization and reinforcement of cohesive soils. Primary reagents are granular or powdered lime for lime columns and cement or lime-cemented mixtures.</td>
</tr>
</tbody>
</table>

Table 14-12 Deep Soil Mixing Groups (Elias et al., 2006)

14.6.1 Analysis

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

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

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

14.6.2.1 Wet Deep Soil Mixing Methods

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

The disadvantages/limitations of wet deep soil mixing are the relative high cost of mobilization of the mixing equipment plus the cost of accompanying auxiliary batch plants. Wet deep soil mixing is uneconomical for small projects. A more extensive geotechnical exploration is required prior to using wet deep soil mixing than is typical. In addition, bench scale testing must be conducted and may require several months to complete. Dense cohesionless soils can not be readily penetrated by the existing deep soil mixing equipment. The amount of spoil produced by deep soil mixing is generally less than for some ground improvement methods. Spoil generation can range from 30 to 100% depending on project specifics, equipment and methods used, and in-situ moisture content. Disposal of this spoil can add significant cost to a project. There is a lack of well developed design and analysis models available. Lastly, there is no standardized method of quality control testing, making design verification difficult and subjective.

14.6.2.2 Dry Deep Soil Mixing Methods

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

One of the disadvantages/limitations of dry deep soil mixing methods is the full strength of the columns may not be mobilized when the pH of the groundwater is acidic or the content of carbon dioxide (CO$_2$) is high. Low strength development should also be anticipated when mixing non-reactive cohesive soils (clays lacking pozzolans). The air-driven injection process may accumulate large quantities of air in the ground potentially causing heave of the adjacent ground surface. This problem can be eliminated by adding mixing paddles to the mixing tool and/or substantially increasing the mixing time. The creep strength of the columns and the shear strength of the stabilized soil is time dependent. Therefore, several months may be required to
perform the laboratory bench scale testing. The average shear strength of the stabilized soil has to be at least three to five times the initial shear strength before dry deep soil mixing becomes economical. There is a lack of well developed design and analysis models available. Lastly, there is no standardized method of quality control testing, making design verification difficult and subjective.

**14.6.3 Feasibility**

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

**14.6.3.1 Site Investigation**

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

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

**14.6.3.2 Assessment**

Deep soil mixing is best used when the subsurface conditions are soft to loose with no obstructions to depths no greater than 100 feet. There should be unrestricted overhead clearance and a need for relatively vibration free ground improvement methods. Deep soil mixing will cause the temporary loss of in-situ soil strength, which may affect adjacent structures. The assessment should review the information obtained from the site investigation. Selected soil chemical properties are provided in the table below.
### Property | Favorable Soil Chemistry |
<table>
<thead>
<tr>
<th></th>
<th></th>
</tr>
</thead>
<tbody>
<tr>
<td>pH</td>
<td>&gt; 5</td>
</tr>
<tr>
<td>Natural moisture content</td>
<td>&lt; 200 (dry method)</td>
</tr>
<tr>
<td>Organic content</td>
<td>&lt; 65 (wet method)</td>
</tr>
<tr>
<td>Loss on Ignition</td>
<td>&lt; 10</td>
</tr>
<tr>
<td>Humus content</td>
<td>&lt; 1</td>
</tr>
<tr>
<td>Electrical conductivity</td>
<td>0.4 mΩ/cm</td>
</tr>
</tbody>
</table>

**Table 14-13 Favorable Soil-Chemistry Factors (Elias et al., 2006)**

#### 14.6.3.3 Preliminary Testing

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

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

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

**Table 14-14 Typical Improved Engineering Properties (Elias et al., 2006)**
Provided in the table below are guidelines related to the penetration, mixing speed, water cement ratio, and reagent content typically used in practice.

<table>
<thead>
<tr>
<th>Work</th>
<th>Guideline</th>
</tr>
</thead>
<tbody>
<tr>
<td>Reagent Content</td>
<td>9 ½ - 22 ½</td>
</tr>
<tr>
<td>Mixing Rotational Speed</td>
<td>20 – 45 rpm</td>
</tr>
<tr>
<td>Penetration Rate</td>
<td>1 yd/min</td>
</tr>
<tr>
<td>Water Cement Ratio</td>
<td>0.6 – 1.3 but 1.0 is normal</td>
</tr>
</tbody>
</table>

Table 14-15 Mixing Guidelines
(Elias et al., 2006)

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

\[
T = N\left[\left(\frac{R_p}{S_p} \times \frac{W_i}{W}\right) + \frac{R_w}{S_w}\right]
\]

where:
- \(N\) = total number of mixing blades
- \(S_p, S_w\) = penetration and withdrawal speed (yard/min)
- \(R_p, R_w\) = blade rotation speed during penetration and withdrawal (rpm)
- \(W_i\) = stabilizer (reagent) injection on penetration (pcf)
- \(W\) = total amount of stabilizer (reagent) (pcf)

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

14.6.4 Design

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

The area replacement ratio \( \alpha_s \) is defined as:

**Equation 14-19**

\[
\alpha_s = \frac{A_s}{A}
\]

where:
- \( \alpha_s \) = area replacement ratio
- \( A_s \) = area of the soil mixed column
- \( A \) = total area within the unit cell

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

**Equation 14-20**

\[
n = \frac{\sigma_s}{\sigma_c}
\]

where:
- \( n \) = stress ratio or stress concentration
- \( \sigma_s \) = stress in the soil mixed column
- \( \sigma_c \) = stress in the surrounding soil

Equilibrium of vertical forces for a given \( \alpha_s \) is provided by the following equation.

**Equation 14-21**

\[
q = \sigma_s \alpha_s + \sigma_c (1 - \alpha_s)
\]

where:
- \( q \) = average stress on the unit cell
The stresses in the soil mixed column and the surrounding soil in the unit cell can be determined by rearranging the above equation.

Equation 14-22

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

Equation 14-23

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

According to *Ground Improvement Methods*, “The total undrained shear resistance \( \tau \) of the stabilized soil is assumed to correspond to the sum of the shear strengths of the column and the soil between the columns and can be evaluated from”

Equation 14-24

\[
\tau = \tau_f \alpha_s + C_u (1 - \alpha_s)
\]

where:
- \( \tau_f \) = undrained shear strength of soil mixed column
- \( C_u \) = undrained shear strength of soil between columns
- \( \alpha_s \) = area replacement ratio

“Typical area replacement ratios are on the order of 0.20 to 0.40 and are varied until the targeted minimum total undrained shear resistance of the stabilized soil is calculated. It is anticipated that area replacement ratios of 0.20 to 0.33, and stress ratios of between 4 and 6, would be used typically for either block- or column-type patterns (see Figure 14-28). The reduction in settlement is attributed to the concept that the soil mixed columns that are stiffer than the adjoining soil will carry more load.”

![Figure 14-28 Deep Soil Mixing Treatment Patterns](Elias et al., 2006)
**14.6.5 Wet Soil Mix Material Properties**

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

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

Table 14-16 Typical Improved Compressive Strength, Wet Mix Method (Elias et al., 2006)

**14.6.6 Dry Soil Mix Material Properties**

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

**14.6.7 Verification**

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

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

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

The definitions contained in the Ground Improvement Methods manual are used in this Manual. The Ground Improvement Methods manual identifies two principle types of grouting which are listed in the table below. Figure 14-29 provides schematics of the various types of grouting.

<table>
<thead>
<tr>
<th>Principle Type of Grouting</th>
<th>Specific Type of Grouting</th>
</tr>
</thead>
<tbody>
<tr>
<td>Rock Grouting</td>
<td>Fissures (using High Mobility Grouts (HMG))</td>
</tr>
<tr>
<td></td>
<td>Voids (natural and artificial, using Low Mobility Grouts (LMG))</td>
</tr>
<tr>
<td>Soil Grouting</td>
<td>Permeation (using HMG and solution grouts)</td>
</tr>
<tr>
<td></td>
<td>Compaction (or displacement)</td>
</tr>
<tr>
<td></td>
<td>Jet (or replacement)</td>
</tr>
<tr>
<td></td>
<td>Fracture (including compensation grouting)</td>
</tr>
</tbody>
</table>

Table 14-17 Types of Grouting Methods
(Elias et al., 2006)
14.7.1 Grout Materials

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

1. Particulate (suspension or cementitious) grout
2. Colloidal 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 14-30 shows the increase in apparent viscosity with time for these grouts and Figure 14-31 shows grain-size distribution of various cements.

![Graph showing Binghamian fluids and viscosity over time](image)

**Figure 14-30 Viscosity vs. Time for Category 1 Grouts**
*(Elias et al., 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 14-32 provides an indication of the change of viscosity with time for these grouts. Category 2 grouts are colloidal solutions that are comprised of mixtures of sodium silicate and a reagent, which when mixed, change viscosity over time to a gel. Sodium silicate is an alkaline, colloidal aqueous solution, while the reagents may be organic or inorganic (mineral). The common types of organic reagents are monoesters, diesters, triesters and aldehydes. These reagents react with the sodium silicate to produce acid as a by-product and can produce either a soft or hard gel depending on the concentration of each compound. The inorganic reagents contain cations that are capable of neutralizing the silicate alkalinity. Typical inorganic reagents are sodium bicarbonate and sodium aluminate. The relative proportions of silicate and reagent will be determined by their own chemistry and concentration, the desired short- and long-term properties, such as gel setting time, viscosity, strength, syneresis and durability, as well as cost and environment acceptability.
Figure 14-32 Viscosity vs. Time for Category 2 and 3 Grouts (Elias et al., 2006)

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

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

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

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

In applications where the durability of the grout is important, resins are typically used for both strength and waterproofing. Resins may be divided into four subcategories as indicated in Table 14-18.

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

**Table 14-18 Types, Use, and Applications of Resins**
(Elias et al., 2006)

There are only two types of polyurethanes appropriate for grouting. These types are listed in Table 14-19.

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

**Table 14-19 Polyurethane Types**
(Elias et al., 2006)
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.

14.7.2 Rock Grouting

14.7.2.1 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. Ground Improvement Methods includes slabjacking (mudjacking) as a subset of rock void grouting. Slabjacking is the process of injecting grout under pressure to raise and relevel concrete paving (typically bridge approach slabs) that have settled (see Figure 14-33).
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 Geotechnical Engineering Bureau for guidance in the use of this method for remediation of scour. As indicated previously, slabjacking is used to correct the settlement of concrete slabs placed over compressible soils or to replace soils that have eroded away from beneath the slab. Typically, this method is used to correct problems associated with the vertical displacement of bridge approach slabs. According to Ground Improvement Methods, “Slabjacking procedures include raising or leveling, under-slab void filling (no raising), grouting slab joints, and asphalt subsealing. Most slabjacking uses a suite of cementitious grouts, incorporating bentonite, sand, ash and/or other fillers, as dictated by local preference and the project conditions and goals. Certain proprietary methods use expanding chemical foams to create uplift pressures. Best results (when no cracking is caused to the slabs) are obtained when the slabjacking is uniformly and gradually conducted. Slabjacking can also be used to “pump” sections of rigid pavements that have sunk below the adjoining section so that the expansion joint may be repaired or have it's functionality restored.”

Figure 14-33 Slabjacking Schematic
(Elias et al., 2006)
Slabjacking has the following advantages:

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

Following are the disadvantages of slabjacking:

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

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

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

### 14.7.3 Soil Grouting

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

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

The general limitation of soil grouting is the soil type to be treated. Although the range of soil grouting available encompasses most soil types, individual methods are limited to specific soils as shown in Figure 14-34.

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

![Figure 14-34 Range of Applicability of Soil Grouting Techniques (Elias et al., 2006)](image)
• Establishing specific objectives for the grouting program (Designer)
• Defining the geometric and geotechnical project conditions (Designer)
• Developing an appropriate grouting program design and compaction specifications and contract documents (Designer)
• Planning the grouting equipment needs and procedural approach (Contractor)
• Monitoring and evaluation of the grouting program (Designer and Contractor)"

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

14.7.3.1 Permeation Grouting

Permeation grouting uses a variety of grout materials, particulate, colloidal and solution, to permeate the soils with little to no disturbance to the original soil structure. The choice of which grout material to use is based on the grain-size distribution of the soil to be grouted (see Figure 14-35). 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 14-20
and Figure 14-36 provide the approximate percentage of material passing the #200 sieve and the groutability of a soil.

![Figure 14-35 Penetrability of Various Grouts vs. Soil Type (Elias et al., 2006)](image)

<table>
<thead>
<tr>
<th>Percent Passing #200 Sieve</th>
<th>Description</th>
</tr>
</thead>
<tbody>
<tr>
<td>&lt; 12</td>
<td>Readily groutable</td>
</tr>
<tr>
<td>12 – 15</td>
<td>Moderately groutable</td>
</tr>
<tr>
<td>15 – 20</td>
<td>Marginally groutable</td>
</tr>
<tr>
<td>&gt; 20</td>
<td>Non-groutable</td>
</tr>
</tbody>
</table>

Table 14-20 Groutability Guidelines
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.

**Equation 14-25**

\[
\frac{D_{15_{soil}}}{D_{85_{grout}}} = \Psi
\]

**Equation 14-26**

\[
\frac{D_{10_{soil}}}{D_{95_{grout}}} = \Theta
\]

where:
- \(D_{15_{soil}}\) = diameter of the 15% passing for soil
- \(D_{85_{grout}}\) = diameter of the 85% passing for the grout material
- \(D_{10_{soil}}\) = diameter of the 10% passing for soil
- \(D_{95_{grout}}\) = diameter of the 95% passing for the grout material

---

**Figure 14-36 Grain-Size Distribution for Permeation Grouting**

(Elias et al., 2006)
<table>
<thead>
<tr>
<th>Groutability</th>
<th>$\Psi$</th>
<th>$\Theta$</th>
</tr>
</thead>
<tbody>
<tr>
<td>Impossible</td>
<td>&lt; 11</td>
<td>&lt; 6</td>
</tr>
<tr>
<td>Possible</td>
<td>11 – 24</td>
<td>6 – 11</td>
</tr>
<tr>
<td>Easy</td>
<td>&gt; 24</td>
<td>&gt; 11</td>
</tr>
</tbody>
</table>

### Table 14-21 Guide to Permeation Grout Potential

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.

#### 14.7.3.2 Compaction Grouting

According to *Ground Improvement Methods*, “Compaction grouting features the use of low slump (usually 1 inch or less), low mobility grouts of high internal friction injected at high grout pressures (up to 600 psi). In weak or loose soils, the grout typically forms a coherent ‘bulb’ at the tip of the injection pipe, thus compacting and/or densifying the surrounding soil. … If settlement has already occurred, careful compaction grouting may be used to lift and level any surface structures that have been impacted. Compaction grouts can be designed as an economic and controllable medium for helping to fill large voids, even in the presence of flowing water.”

Compaction grouting has a wide variety of applications, but is primarily used for soil densification (for both static and seismic enhancements) and for raising surficial structures. In soil densification applications, the soils should be free-draining, such as gravels, relatively clean sands and some coarser silts (see Figure 14-36). In fine-grained soils, pore pressures may not be able to dissipate and improvement may not be achievable. In these soils, compaction grouting may displace the soil, but not cause settlement or consolidation.

The grout mix design is a critical part of compaction grouting; the grout must have a high internal friction and a low slump to ensure the “bulb” forms. There are no mathematical models for use in compaction grouting (i.e., establishing the spacing, rate of injection, limiting volumes, etc.). Therefore, either an Engineer or Contractor that specializes in compaction grouting should be retained to assist in the final design of compaction grouting. Typically compaction grout pipes are spaced at 6 ½ to 16 ½ feet intervals. The amount of grout required for soil densification ranges from three to twelve percent of the soil volume being treated. Normally, compaction grouts use particulate grouts such as Portland Cement Types I or II. The preferred gradation for the sand is shown in Figure 14-37. The slump of the compaction grout should be around 1 inch.
Jet grouting is a grouting process that uses very high pressure (up to 7000 psi), high velocity erosive jets of water and/or grout to remove and loosen soil, replace the removed soil with cement based grout, and then mix the grout into the loosened soil to form a grouted column of soil. 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 14-38 provides a schematic of the jet grouting procedure.
Jet grouting can be accomplished using three different types of jetting procedures as discussed below and depicted in Figure 14-39.

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

Soil fracture grouting is the process of injecting grouts in a highly controlled manner that does not permit permeation of the grout in the soil matrix nor compaction of the soil matrix. Instead the soil matrix is ruptured and kept separated through high pressure and injection rates 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.

14.8 COLUMN SUPPORTED EMBANKMENT

Constructing embankments over soft, compressible soils creates numerous problems (i.e., excessive settlements, embankment instability, and long times for settlements to occur). These problems have led to the development of the ground improvement methods discussed previously in this Chapter; however, in certain cases, time constraints are critical to the success of the project. Therefore, an alternate ground improvement method has been used: Column Supported Embankment (CSE) (see Figure 14-40). CSEs consist of two primary components; first, a column system to transfer loads to a more suitable bearing stratum and second, a load transfer platform (LTP). The LTP can consist of either structural concrete or a geosynthetic reinforced soil layer.
Figure 14-40 CSE with Geosynthetic LTP  
(Elias et al., 2006)

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

The LTP transfers the embankment load to the columns. The LTP may consist of either a rigid structural element or a geosynthetic reinforced soil layer. The rigid LTP is typically economically cost prohibitive and will therefore, not be discussed in this Chapter. The use of a LTP allows for the columns to be more widely spaced; however, the use of an LTP is not required if the columns are closely spaced, see Figure 14-41.
14.8.1 Analysis and Preliminary Design

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

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

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

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

14.8.2 Design

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

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

The Strength limit state failure modes include the following (see Figure 14-42):

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 14-42 Strength Limit State Failure Modes  
(Elias et al., 2006)
The Service limit state of the CSE must also be checked. The strain in the geosynthetic reinforcement used to create the LTP should be kept below some maximum threshold to preclude unacceptable deformation reflection (see Figure 14-43, 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 14-43, Detail b).

![Figure 14-43 Service Limit State](Elias et al., 2006)

The general design procedure for CSEs is provided below:

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

14.8.2.1 Column Design

The selection of the type of column should be based on the constructability, load capacity, and cost of the various column types. The load carrying capacity of each column is based on the tributary area of each column (see Figure 14-44). In CSE design, it is assumed that the weight of the embankment and any surcharge loads are carried by the columns and that the surrounding soil carries minimal, if any, load. The tributary area for a single column is geometrically a hexagon; however, for simplification a circle having the same tributary area is used. Figure 14-44 provides the effective diameter (D_e) for both triangular and square spacings. The typical center-to-center column spacing is 5 to 10 feet. The required design vertical load (Q_r) in the column is determined by the following equation:

Equation 14-27

\[ Q_r = \pi \left( \frac{D_e}{2} \right)^2 (\gamma H + q) \]

where:
\( D_e \) = effective tributary area of column
\( H \) = height of embankment
\( \gamma \) = unit weight of embankment soil
\( q \) = live and dead load surcharge (determined similar to long-term stability analysis)
In addition to determining the load to be carried by the columns, the lateral extent of the columns will also need to be determined. The columns should extend a sufficient distance beyond the crest of the embankment to ensure that any instability or differential settlement that occurs beyond the limits of the columns will not affect the crest of the embankment. The extent of the columns should be determined using a slope stability program. For preliminary designs and feasibility studies, the following equations from the British Standard (BS8006) may be used.

**Equation 14-28**

\[ L_p = H(n - \tan \theta_p) \]
Equation 14-29

\[ \theta_p = (45 - \frac{\phi'_{emb}}{2}) \]

where:
- \( L_p \) = horizontal distance from the toe of the embankment to the edge of the first column
- \( n \) = side slope of embankment (see Figure 14-45)
- \( \theta_p \) = angle from vertical between the outer-most column and the crest of the embankment (see Figure 14-45)
- \( \phi'_{emb} \) = effective friction angle of embankment fill

The potential for lateral spreading of the embankment must be analyzed (Figure 14-45). The geosynthetic reinforcement must be designed to prevent lateral spreading of the embankment. This is a critical aspect of the design, because many columns used to support CSEs are not capable of developing adequate lateral resistance to prevent the spreading of the embankment. The geosynthetic reinforcement must be designed to resist the horizontal force caused by the lateral spreading of the embankment. The required tensile force to prevent lateral spreading (\( T_{ls} \)) is determined using the following equations.
Equation 14-30

\[ T_{ls} = \frac{K_a(\gamma H + q)H}{2} \]

Equation 14-31

\[ K_a = \tan^2\left(45^\circ - \frac{\phi'_{emb}}{2}\right) \]

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

Equation 14-32

\[ L_e = \frac{T_{ls}}{[0.5\gamma H(c_{iemb} \tan \phi'_{emb})]} \]

where:
- \( c_{iemb} \) = coefficient of interaction for sliding between the geosynthetic reinforcement and the embankment fill
- \( \phi'_{emb} \) = friction angle of embankment fill material (\( \phi'_{cv} \) in Figure 14-46)

Figure 14-46 CSE Lateral Spreading (Elias et al., 2006)
14.8.2.2 Load Transfer Platform

The load transfer platform (LTP) has two design approaches (catenary or beam) (see Figure 14-47). The three methods for design of the LTP using the catenary design approach are the British Standard (BS 8006), the Swedish Method and the German Method. The catenary approach makes the following assumptions:

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

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

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

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

Figure 14-47 Load Transfer Mechanisms (Elias et al., 2006)
Both design approaches rely on the soil forming an arch. Soil arching, according to Ground Improvement Methods, is “the ability of material to transfer loads from one location to another in response to a relative displacement between locations.” Figure 14-48 provides schematics of soil arching.

According to Ground Improvement Methods, “The stress at point “a” in Figure 14-48, Detail a, is equal to the overburden stress $\gamma H$, where $\gamma$ is the unit weight of the soil, and $H$ is the height of the soils mass. At the moment when the soil loses support, a temporary true arch is formed. The soil at point “a” is in tension, and the weight of the soil prism starts to be transferred to the adjacent unyielding soil (Figure 14-48, Detail b). Deformation within the temporary true soil arch occurs. As the soil settles into an inverted arch (Figure 14-48, Detail c), an equilibrium state is achieved, the adjacent unyielding soil mobilizes its shear strength, and the load transfer is complete. At some height ($H_e$) above point “a,” the transfer of stress is complete. The settlements in the soil mass above this point are uniform. The degree of soil arching is defined as the soil arch ratio ($\rho$), which is the ratio of the average vertical stress on the yielding portion (i.e., soft soil between columns) to the average vertical stress due to the embankment fill and surcharge load.”

Equation 14-33

$$\rho = \frac{\sigma_s}{(\gamma_{emb} H + q)}$$

where:

- $H$ = height of embankment
- $\gamma_{emb}$ = unit weight of embankment material
- $q$ = live and dead load surcharge (determined similar to long-term stability analysis)
- $\sigma_s$ = average vertical stress applied between columns
Figure 14-48 Soil Arching
(Elias et al., 2006)
In addition to soil arching, the load transfer platform design relies on the geosynthetic developing resistance to tension (i.e., tension membrane theory). The vertical load from the soil within the arch and any surcharge load, if the thickness of the embankment is not great enough to develop the full arch, is carried by the reinforcement.

The variables and symbols used by all of the design methods have been standardized. Figure 14-49 depicts the common symbols that will be used in presenting each of these methods; further each variable and/or symbol is defined also.

Where,

\( d \) = Diameter of column  
\( H \) = Height of embankment  
\( P_c' \) = Vertical stress on column  
\( q \) = Surcharge load  
\( s \) = Center-to-center column spacing  
\( TRP \) = Tension in the extensible reinforcement  
\( WT \) = Vertical load carried by the reinforcement  
\( \gamma_{emb} \) = Unit weight of embankment material  
\( \phi'_{emb} \) = Effective friction angle of the embankment fill material
14.8.3 Catenary Design Approach

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

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

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

Equation 14-34

\[
\frac{P_c'}{\sigma_c} = \left( \frac{C_c d}{H} \right)^2
\]

Equation 14-35

\[
\sigma_c' = (f_{fs} \gamma_{emb} H + f_q q)
\]

where:

- $\sigma_c'$ = average vertical stress at the base of the embankments
- $f_{fs}$ = partial soil unit mass load factor (1.3)
- $f_q$ = partial surcharge load factor (1.3)
- $C_c$ = arching coefficient
  - $= [(1.95H/d)-0.18]$ for end bearing columns (unyielding)
  - $= [(1.50H/d)-0.07]$ for frictional columns (normal)

The vertical load ($W_T$) carried by the reinforcement is determined from the following equations:
**Table 14-22 Vertical Load Determination Equations**

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

**Equation 14-38**

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

where:

- $\varepsilon = \text{strain in the reinforcement}$

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

**14.8.3.2 Swedish Method**

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

- Arch formation occurs
- Reinforcement is deformed during loading
- One layer of reinforcement is used
- Reinforcement is located within 4 inches of the column
- The embankment height is greater than or equal to the clear distance between the columns (edge-to-edge)
- The ratio of column area to the influence area per column is greater than or equal to ten percent
- The embankment fill effective friction angle is 35°
- Initial strain in the reinforcement is limited to six percent
- Long-term (creep) strain is limited to two percent
- Total strain is less than seventy percent strain at failure

The model used in the Swedish Method to determine the vertical load is provided in Figure 14-50. The cross sectional area of the soil under the arch, which is the load carried by the reinforcement, is approximated by the soil wedge shown in Figure 14-50. This applies even when the embankment height is lower than the top of the soil wedge. The height of the soil wedge is determined from the following equation:

**Equation 14-39**

\[ h = \frac{(s - d)}{2 \tan 15°} \]

where:
- \( h \) = height of soil wedge or arch
- \( s \) = center-to-center column spacing
- \( d \) = diameter of column

**Figure 14-50 Swedish Method Model (Rogbeck et al., 2004)**
The two-dimensional weight ($W_T$) of the soil wedge per unit length in depth is determined from the following equation.

**Equation 14-40**

$$W_T = \frac{(s - d)^2 \gamma_{emb}}{4 \tan 15^\circ}$$

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

**Equation 14-41**

$$T_{RP} = 0.5W_T[1 + (\frac{s}{d})^2]\sqrt{1 + \frac{1}{6E}}$$

### 14.8.3.3 German Method

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

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

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

**Equation 14-42**

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

**Equation 14-43**

$$A = [1 - (\frac{d}{s})^{2(K_p - 1)}]$$
Equation 14-44

\[ B = \left[ \frac{s}{1.41H} \right] \left[ \frac{(2K_p - 2)}{(2K_p - 3)} \right] \]

Equation 14-45

\[ C = \left[ \frac{(s - d)}{1.41H} \right] \left[ \frac{(2K_p - 2)}{(2K_p - 3)} \right] \]

Equation 14-46

\[ K_p = \frac{(1 + \sin \phi_{emb})}{(1 - \sin \phi_{emb})} = \tan^2 \left( 45 + \frac{\phi_{emb}}{2} \right) \]

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

Equation 14-47

\[ E = \frac{\beta}{(1 + \beta)} \]

Equation 14-48

\[ \beta = \left[ \frac{2K_p}{(K_p + 1)(1 + \frac{d}{s})} \right] \left[ \left( 1 - \frac{d}{s} \right)^{-\kappa_p} - \left( 1 + \frac{K_p d}{s} \right) \right] \]

The minimum value of E controls the stress applied to the soil between the columns (\( \sigma_s \)). The stress that is applied to the soil between columns (Figure 14-51) is determined using the following equation.

Equation 14-49

\[ \sigma_s = \left[ \frac{\gamma_{emb} H + q}{(s^2 - d^2)} \right] \left[ 1 - E \right] s^2 \]
The geosynthetic reinforcement carries the stress imposed by the embankment ($\sigma_s$) minus the resistance of the soil ($\sigma_o$) located between the columns. The resistance of the soil ($\sigma_o$) between the columns is determined using the following equation.

**Equation 14-50**

$$\sigma_o = 0.5[(2 + \pi)c_u]$$

where:

$c_u$ = undrained shear strength of the soil between the columns
0.5 = resistance factor ($\phi$) used to determine $\sigma_o$

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

**Equation 14-51**

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

where:

$s' = s$ for square column pattern
$s' = 1.4s$ for triangular column pattern
The German Method assumes that the geosynthetic reinforcement is placed less than 1 ½ feet above the columns. The tensile force in the reinforcement per unit length of reinforcement is determined based on catenary tension and is determined using the following equation.

**Equation 14-52**

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

**14.8.4 Beam Design Approach**

The beam design approach consists of one method, the Collin Method, and is fundamentally different from the catenary design approach. The beam design approach is based on the premise that the reinforcement creates a stiffened beam of reinforced soil to distribute the load imposed by the embankment to the columns. The stiffened beam of reinforced soil should contain a minimum of three layers of reinforcement (Figure 14-52). This beam acts as the LTP for the Collin Method.

The Collin Method is based on the following assumptions:

- The thickness (h) of the LTP is equal to or greater than one-half of the clear span between the columns (i.e., 0.5(s-d))
- A minimum of three layers of geosynthetic reinforcement is used to create the LTP
- A minimum distance of 8 inches is maintained between the layers of reinforcement
- Select fill is used to construct the LTP
- The primary function of the reinforcement is to provide lateral confinement of the select fill to facilitate soil arching within the thickness (h) of the LTP
- The secondary function of the reinforcement is to support the wedge of the soil below the arch
- All of the vertical load from the embankment above the load transfer platform is transferred to the columns below the platform
- The initial strain in the reinforcement is limited to five percent
Figure 14-52 Load Transfer Platform (Elias et al., 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 14-53). 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.

**Equation 14-53**

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

**Equation 14-54**

$$W_{Tn} = \frac{(s - d)_n^2 + (s - d)_{n+1}^2) n \gamma_{emb}}{(s - d)_n^2}$$
The tensile load on any layer of reinforcement ($T_{R_{P_n}}$) is determined based on tension membrane theory and is a function of the amount of strain in the reinforcement. $T_{R_{P_n}}$ is determined using the following equation:

**Equation 14-55**

$$T_{R_{P_n}} = W_{T_n} \Omega D / 2$$

where:
- $D = (s-d)_n$ for square column spacing
- $D = (s-d)_n \tan 30$ for triangular column spacing
- $\Omega = \text{from Table 14-23}$
<table>
<thead>
<tr>
<th>Ω</th>
<th>Reinforcement Strain (ε) %</th>
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</tr>
<tr>
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<tr>
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<td>4</td>
</tr>
<tr>
<td>0.97</td>
<td>5</td>
</tr>
</tbody>
</table>

Table 14-23 Values of Ω
(Elias et al., 2006)

14.8.5 Reinforcement Total Design Load

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

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

**Equation 14-56**

\[ T_{\text{max}} = T_{RP} \]

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

**Equation 14-57**

\[ T_{\text{max}} = T_{RP} + T_{ls} \]
14.9 REFERENCES


Moore, L.H., and Grosert, T., *An Appraisal of Sand Drain projects Designed and Constructed by the New York State Department of Transportation*, NYSDOT Physical Research Project No. 5.

sand deposits with vertical drains, UCB/EERC-97/15, Earthquake Engineering Research Center, University of California, Berkeley, 1997.


