CHAPTER 26

Soil Improvement

Soil improvement is an alternative considered when the natural soil does not meet the engineering requirements for a project. As an example, if the soil is too weak to carry the structure on a shallow foundation, two alternatives exist: deep foundations or soil improvement plus a shallow foundation. A soil improvement technique is sought that would make a shallow foundation feasible. If the deep foundation will cost $1,000,000, while the soil improvement will cost $250,000 and the shallow foundation $500,000, then the soil improvement alternative becomes attractive. Typically in this case the soil improvement technique is verified by in situ testing to demonstrate that a sufficiently improved soil strength and soil modulus can be reached so that a shallow foundation is viable.

A very large number of methods are aimed at soil improvement; this chapter summarizes the main methods. For additional information, the following excellent references can be consulted: the state-of-the-art report, published by the ISSMGE Technical Committee on ground improvement and presented at the 2009 International Conference on Soil Mechanics and Geotechnical Engineering (Chu et al. 2009); the book by Moseley and Kirsch (2004); the NHI manual (Elias et al. 2006); and the web site www.geotechtools.org (Schaefer 2013).

26.1 OVERVIEW

Over the past 50 years, many different soil improvement techniques have been developed, and they continue to be developed and revised as the space available for human activities decreases. These methods have been classified by the ISSMGE Technical Committee on Ground Improvement as shown in Table 26.1. The word *ground* is used in that classification because it can incorporate rock, but because this book is limited to soil, the term *soil improvement* is used here. There are five major categories of soil improvement methods.

1. Soil improvement without admixture in coarse-grained soils
2. Soil improvement without admixture in fine-grained soils
3. Soil improvement with replacement
4. Soil improvement with grouting and admixtures
5. Soil improvement with inclusions

26.2 SOIL IMPROVEMENT WITHOUT ADMIXTURE IN COARSE-GRAINED SOILS

26.2.1 Compaction

Compaction in this instance refers to roller compaction for shallow densification of soil deposits. The rollers used are static rollers, such as sheep-foot rollers for fine-grained soils or vibratory rollers for coarse-grained soils. Most rollers are cylindrical, but some are uneven rollers. The depth of compaction is at most 1 m and is highest near the surface. Compaction is used to prepare pavement layers, retaining wall backfills, and embankment fills. This topic is covered in Chapter 20.

26.2.2 Dynamic Compaction

Because of the limited depth of conventional compaction techniques and the need to compact natural soils at larger depths, the idea of dropping a heavy weight from a height onto the soil surface was pioneered by Louis Menard (Menard and Broise 1975). A typical combination would be a 20-ton weight dropping from a height of 20 m. This technique is best suited to compaction of coarse-grained soils. This topic, including the depth that can be reached and the improvement ratio versus depth, is covered in Chapter 20.

26.2.3 Vibrocompaction

The vibrocompaction method consists of lowering a cylindrical vibrator from a crane into the soil to densify the soil (Figure 26.1). A grid of 3 to 4 meters center to center is common. The vibrator is 2 to 5 m long and 0.3 to 0.5 m in diameter, and weighs 15 to 40 kN. The vibrations are generated in the horizontal direction by rotating eccentric masses. The frequency of vibration is in the range of 25 to 35 Hz with amplitudes between 10 to 30 mm. The vibrator typically

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Table 26.1 Classification of Soil Improvement Methods (Chu et al. 2009)

Category	Method	Principle
A. Ground improvement without admixtures in noncohesive soils or fill materials	A1. Dynamic compaction	Densification of granular soil by dropping a heavy weight from air onto ground.
	A2. Vibrocompaction	Densification of granular soil using a vibratory probe inserted into ground.
	A3. Explosive compaction	Shock waves and vibrations generated by blasting cause granular soil ground to settle through liquefaction or compaction.
	A4. Electric pulse compaction	Densification of granular soil using the shock waves and energy generated by electric pulse under ultra-high voltage.
	A5. Surface compaction (including rapid impact compaction)	Compaction of fill or ground at the surface or shallow depth using a variety of compaction machines.
B. Ground improvement without admixtures in cohesive soils	B1. Replacement, displacement (including load reduction using lightweight materials)	Remove bad soil by excavation or displacement and replace it by good soil or rocks. Some lightweight materials may be used as backfill to reduce the load or earth pressure.
	B2. Preloading using fill (including the use of vertical drains)	Fill is applied and removed to preconsolidate compressible soil so that its compressibility will be much reduced when future loads are applied.
	B3. Preloading using vacuum (including combined fill and vacuum)	Vacuum pressure of up to 90 kPa is used to preconsolidate compressible soil so that its compressibility will be much reduced when future loads are applied.
	B4. Dynamic consolidation with drainage (including the use of vacuum)	Similar to dynamic compaction except that vertical or horizontal drains (or together with vacuum) are used to dissipate pore pressures generated in soil during compaction.
	B5. Electro-osmosis or electrokinetic consolidation	DC current causes water in soil or solutions to flow from anodes to cathodes installed in soil.
	B6. Thermal stabilization using heating or freezing	Change the physical or mechanical properties of soil permanently or temporarily by heating or freezing the soil.
	B7. Hydro-blasting compaction	Collapsible soil (loess) is compacted by a combined wetting and deep explosion action along a borehole.
C. Ground improvement with admixtures or inclusions	C1. Vibro replacement or stone columns	Hole jetted into soft, fine-grained soil and backfilled with densely compacted gravel or sand to form columns.
	C2. Dynamic replacement	Aggregates are driven into soil by high-energy dynamic impact to form columns. The backfill can be either sand, gravel, stones, or demolition debris.
	C3. Sand compaction piles	Sand is fed into ground through a casing pipe and compacted by vibration, dynamic impact, or static excitation to form columns.
	C4. Geotextile confined columns	Sand is fed into a closed-bottom, geotextile-lined cylindrical hole to form a column.
	C5. Rigid inclusions	Use of piles, rigid or semirigid bodies, or columns that are either premade or formed in situ to strengthen soft ground.
	C6. Geosynthetic-reinforced column or pile-supported embankment	Use of piles, rigid or semirigid columns/inclusions, and geosynthetic girds to enhance the stability and reduce the settlement of embankments.
	C7. Microbial methods	Use of microbial materials to modify soil to increase its strength or reduce its permeability.
	C8. Other methods	Unconventional methods, such as formation of sand piles using blasting, and the use of bamboo, timber, and other natural products.

(*Continued*)

Table 26.1 (*Continued*)

Category	Method	Principle
D. Ground improvement with grouting-type admixtures	D1. Particulate grouting	Grout granular soil or cavities or fissures in soil or rock by injecting cement or other particulate grouts to either increase the strength or reduce the permeability of soil or ground.
	D2. Chemical grouting	Solutions of two or more chemicals react in soil pores to form a gel or a solid precipitate to either increase the strength or reduce the permeability of soil or ground.
	D3. Mixing methods (including premixing or deep mixing)	Treat the weak soil by mixing it with cement, lime, or other binders in situ using a mixing machine or before placement.
	D4. Jet grouting	High-speed jets at depth erode the soil and inject grout to form columns or panels.
	D5. Compaction grouting	Very stiff, mortar-like grout is injected into discrete soil zones and remains in a homogenous mass to densify loose soil or lift settled ground.
	D6. Compensation grouting	Medium- to high-viscosity particulate suspensions are injected into the ground between a subsurface excavation and a structure to negate or reduce settlement of the structure due to ongoing excavation.
E. Earth reinforcement	E1. Geosynthetics or mechanically stabilized earth (MSE)	Use of the tensile strength of various steel or geosynthetic materials to enhance the shear strength of soil and stability of roads, foundations, embankments, slopes, or retaining walls.
	E2. Ground anchors or soil nails	Use of the tensile strength of embedded nails or anchors to enhance the stability of slopes or retaining walls.
	E3. Biological methods using vegetation	Use of the roots of vegetation to create and improve stability of slopes.

Figure 26.1 Example of vibrocompactor. (Courtesy of Earth Tech, LLC.)

reaches depths of 20 to 30 m, with 60 m being rare. Pipes go through the body of the vibrator and can supply water or air to the bottom of the vibrator to help with penetration if necessary.

The soils best suited to use of this technique are clean sands. If the fine content becomes higher than 10 to 15%, the vibrocompaction process becomes much less efficient (Mitchell and Jardine 2002). Massarsch (1991) proposed a CPT-based chart indicating which soils are most applicable to vibrocompaction (Figure 26.2).

26.2.4 Other Methods

Other compaction methods include rapid impact compaction (Figure 26.3), explosive compaction, and electric pulse compaction. In rapid impact compaction (Watts and Charles 1993), a tamper is pounded repeatedly on the ground surface. The weight is lifted about 1 m up in the air and dropped at a rate of around 40 drops per minute. The hammer weighs about 100 kN and has a diameter between 1.5 and 1.8 m. This technique is best for sands and gravels and is not suited for saturated silts and clays.

Explosive compaction consists of setting a series of detonation charges in the deposit. These detonations create waves that propagate in the soil and compact it. This technique is not commonly used, but has the advantage of being relatively inexpensive. Electric pulse compaction consists of lowering a probe into the soil and discharging high voltage sparks at a rate of about 10 per minute. This recent method is as yet unproven.

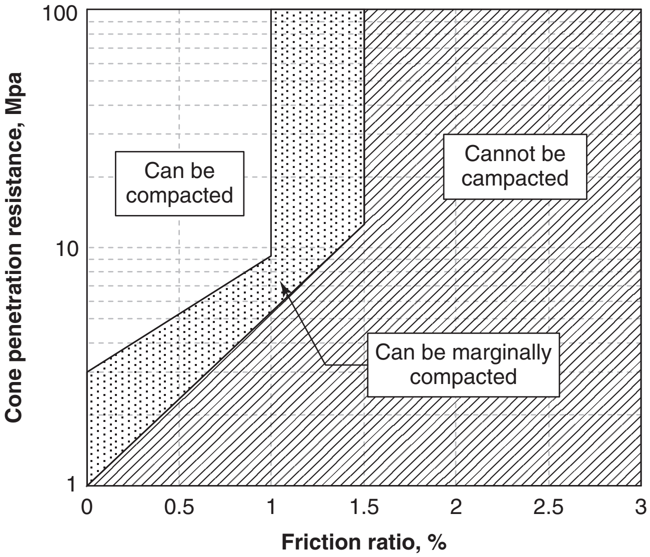


Figure 26.2 Soil suitability for vibrocompaction based on CPT. (Courtesy of Dr. Rainer Massarsch)

Figure 26.3 Example of rapid impact compactor. (Courtesy of Menard Bachy, Inc.)

26.3 SOIL IMPROVEMENT WITHOUT ADMIXTURE IN FINE-GRAINED SOILS

26.3.1 Displacement–Replacement

The displacement-replacement technique consists of simply excavating the weak soil (say, $s_u < 20\,\text{kPa}$) and replacing it with stronger soil. Excavation depths beyond 8 m are uncommon; the method can be costly and environmentally unfriendly because of the amount of spoil to be disposed of. In the case of peat bogs, the backfill, which may be twice as heavy as the natural soil, can create very large settlements. Sometimes the backfill is made of lightweight material such as geofoam blocks (see section 25.3.5) to avoid excessive settlement and bearing capacity issues.

26.3.2 Preloading Using Fill

The technique of preloading using fill consists of loading the soil surface with a fill, as in the case of an embankment and a surcharge fill. It has been used for many years to shorten the time required to reach a certain settlement under the design load. Once the settlement is reached, the surcharge is withdrawn and the road can be paved, for example. It is important to note, in this respect, that the time t_U to reach U percent of consolidation depends not on the height of the fill but on the drainage length H and the soil coefficient of consolidation c_v. In other words, if it takes 5 years to reach 90% of the final settlement under a 5 m high embankment, it will also take 5 years to reach 90% of the final settlement under a 10 m embankment. However, if it takes 5 years to reach 90% of the final settlement under a 5 m high embankment, it will take a lot less time to reach that same settlement under a 10 m high embankment. To find out what height h_s must be added as a surcharge on top of an h_e high embankment to reach, say, 90% of the settlement of the embankment within a target time t_t, use the following steps (Figure 26.4):

1. Calculate the maximum settlement of the embankment $s_{\max(emb)}$. For a normally consolidated clay, the following equation can be used (see section 17.8.9):

$$s_{\max(emb)} = h_o \frac{C_c}{1+e_o} \log\left(\frac{\sigma'_{ov} + \Delta\sigma'}{\sigma'_{ov}}\right) \qquad (26.1)$$

   where h_o is the height of the soft clay layer, C_c is the compression index from consolidation tests, e_o is the initial void ratio of the soft clay layer, σ'_{ov} is the initial effective stress in the middle of the soft clay layer, and $\Delta\sigma'$ is the increase in stress in the middle of the soft clay layer.
2. Choose the target time t_t to reach $s_{\max(emb)}$.
3. Knowing the target time t_t and the coefficient of consolidation c_v of the soft clay layer, calculate the time factor T_U corresponding to t_t using the equation (see section 17.8.10):

$$T_U = \frac{t_t c_v}{h_d^2} \qquad (26.2)$$

   where h_d is the drainage length.

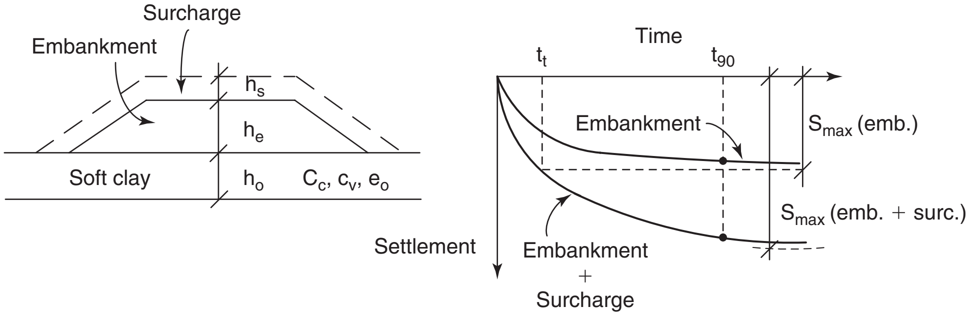


Figure 26.4 Surcharge to accelerate embankment settlement.

This drainage length is equal to the soft clay layer thickness if the water can only drain through the top or the bottom of the layer; equal to one-half of the layer thickness if the water can drain through the top and the bottom of the layer; and equal to the horizontal distance between vertical drains if such drains are installed.

4. Then find the average percent consolidation U corresponding to the time factor T_U by using the curve that links both parameters (Figure 26.5). Note that U is equal to:

$$U = \frac{s(t)}{s_{\max}} \tag{26.3}$$

where $s(t)$ is the settlement after a time t and $s_{\max}$ is the final settlement.

5. Knowing U, use Eq. 26.3 to calculate the maximum settlement $s_{\max(emb+surch)}$ under the embankment plus the surcharge. In Eq. 26.3, U is known and $s(t)$ is equal to the settlement under the embankment plus the surcharge after a time equal to the target time $s(t_t)$. By design, this settlement is equal to the maximum settlement under the embankment only, $s_{\max(emb)}$:

$$s(t_t) = s_{\max(emb)} \tag{26.4}$$

$$s_{\max(emb+surch)} = \frac{s_{\max(emb)}}{U} \tag{26.5}$$

6. Once the maximum settlement under the embankment and the surcharge $s_{\max(emb+surch)}$ is known, Eq. 26.1 can be used to back-calculate the value of $\Delta\sigma'$ induced by the surcharge:

$$\Delta\sigma' = \sigma'_{ov}\left(10^{\left(\frac{(1+e_o)s_{\max(emb+surch)}}{h_o C_c}\right)} - 1\right) \tag{26.6}$$

7. Finally, the height of the surcharge h_s is the height that generates an increase in effective stress in the soft clay layer equal to $\Delta\sigma'$. Often, if the soft clay layer is not very thick compared to the width of the embankment, the increase in stress is equal to the pressure generated by the surcharge at the ground surface and the height of the surcharge is:

$$h_s = \frac{\Delta\sigma'}{\gamma_s} \tag{26.7}$$

where γ_s is the unit weight of the surcharge soil.

8. Note that if the surcharge is too high, a slope stability or bearing capacity problem arises for the side of the

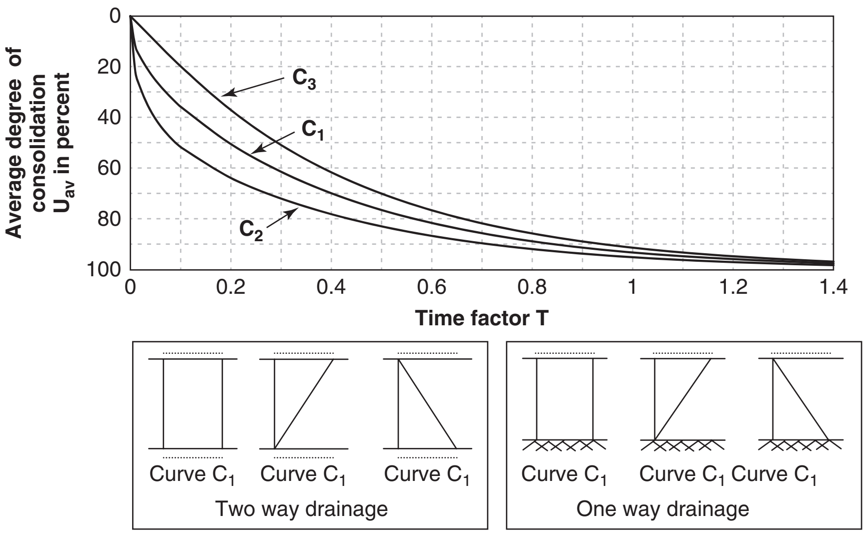


Figure 26.5 Average percent consolidation U versus time factor T_U.

embankment. In that regard, the height of the surcharge $h_{s\,max}$ that would generate a bearing capacity failure in a clay of undrained shear strength s_u can be estimated by:

$$h_{s\,max} = \frac{5.14 s_u}{\gamma_s} \tag{26.8}$$

26.3.3 Prefabricated Vertical Drains and Preloading Using Fill

The technique of using vertical drains and preloading consists of loading the soil surface with a fill while accelerating the consolidation process by installing prefabricated vertical drains (PVDs) or sand drains. Prefabricated vertical drains are also called *wick drains* or *band drains*. They are installed to decrease the drainage length h_d, thereby reducing the time necessary for the consolidation settlement to take place. The drainage is then shifted from a vertical drainage problem involving the vertical hydraulic conductivity k_v to a horizontal drainage problem between PVDs involving the horizontal hydraulic conductivity k_h. For example, if a soft clay layer is 10 m thick and has one-way drainage, the drainage length will be 10 m and the time required for 90% of the final settlement will be t_1. If PVDs are installed on a grid with a center-to-center spacing equal to 2 m, the drainage length is controlled by the horizontal spacing and becomes much shorter, so the time t_2 for 90% of the final settlement is dramatically reduced compared to t_1. Note that the ratio of the two times requires comparing the solution of the one-dimensional consolidation problem for the embankment on top of the layer without PVDs (see section 11.4.6) to the solution for the drainage around a grid of drains; this is often approximated by the radial consolidation problem (Moseley and Kirsch 2004).

PVDs are typically made of a filter material covering both sides of a corrugated plastic shell (Figure 26.6). The width may be 100 mm, the thickness 3 to 4 mm, and the installed length can be 30 m. The flow rate out of such drains is in the range of 2 to 8 liters per minute, but may decrease with time because of siltation, for example. At the same time, the actual flow through the PVD decreases with time as consolidation takes place. Installation of PVDs is done by tying one end of the PVD with an anchor inside a small-diameter pipe called a *mandrel* and pushing the mandrel vertically into the soil and dragging the PVD with it (Figure 26.7). Once at the required depth, the mandrel is withdrawn and the PVD is left in place. PVDs can be placed to depths of several tens of meters on a grid with spacing in the range of 1 to 2.5 m. The tops of the drains are bound by a drainage layer or drainage blanket (0.5 to 1 m thick) made of clean sand, and the water is pumped away from the site. The drainage blanket is often placed before the PVDs are installed and serves as a work platform for the equipment. One issue associated with the placement of PVDs is the development of a "smear zone" in soft clays at the boundary between the soil and the PVD. This smear zone

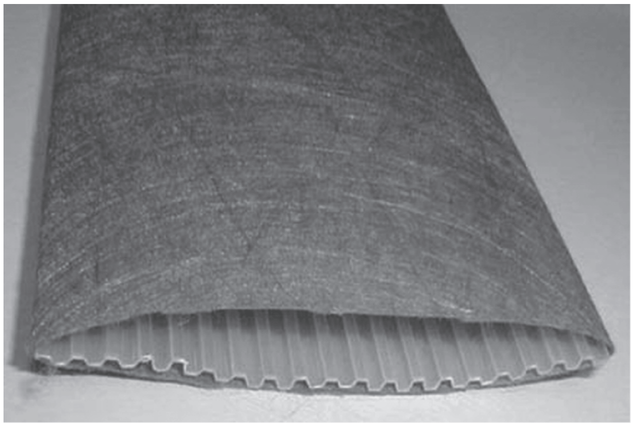
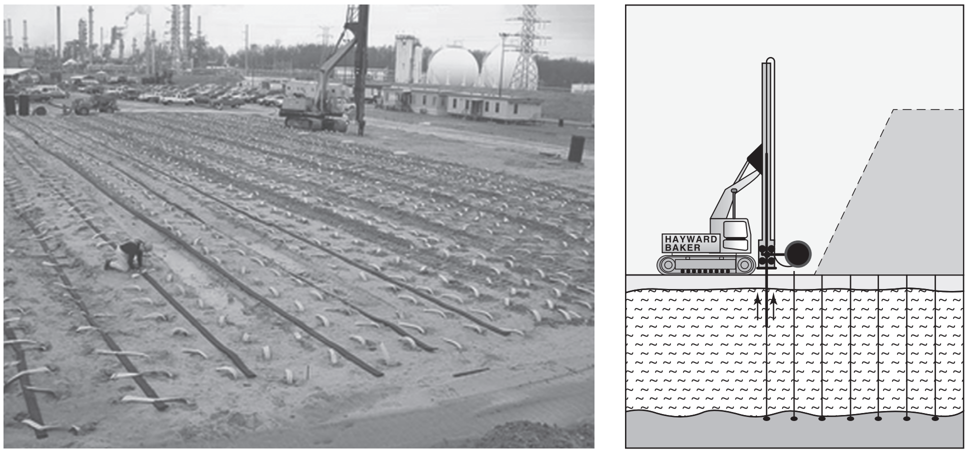


Figure 26.6 Prefabricated vertical drain. (Courtesy of Layfield Environmental Systems, Layfield Group Limited, 11120 Silversmith Place, Richmond, British Columbia, Canada V7A 5E4.)



(a) (b)

Figure 26.7 Installation of prefabricated vertical drains. (Courtesy of Hayward Baker Geotechnical Construction.)

is a few PVD diameters thick and can reduce the permeability of the interface.

The purpose of PVDs is to minimize the consolidation time t needed to reach a given percent consolidation U_h taken as a ratio, not a percent, in Eq. 26.9. This time t can be calculated by using the Barron-Hansbo formula (Barron 1948; Hansbo 1981):

$$t = \frac{d_w^2}{8c_h}\left(Ln\left(\frac{d_w}{d_e}\right) - 0.75 + F_s\right)Ln\left(\frac{1}{1-U_h}\right) \quad (26.9)$$

where d_e is the equivalent diameter of the PVD defined in Eq. 26.10, c_h is the horizontal coefficient of consolidation, d_w is the well influence diameter (taken as 1.05 s for an equilateral triangle spacing pattern and 1.13 s for a square spacing pattern) where s is the spacing between PVDs, and F_s is a soil disturbance factor (taken as 2 for highly plastic sensitive soils but taken as zero if c_h has been conservatively estimated or accurately measured):

$$d_e = \frac{2(a+b)}{\pi} \quad (26.10)$$

where a is the PVD thickness and b is the PVD width.

26.3.4 Preloading Using Vacuum

Sometimes the soil is so soft that a surcharge fill cannot be placed to a sufficient height to be useful. In this case, preloading by vacuum is an alternative. The method consists of applying a vacuum, thereby decreasing the water stress, increasing the effective stress, and compressing the soil. A vacuum of 0.8 atmosphere is commonly applied and is equal to about 4 m of soil surcharge. One difference between this method and a fill method is that for vacuum preloading, the increase in effective stress is applied isotropically, as opposed to anisotropically for the fill. Figure 26.8 shows the difference in effective stress path between a surcharge fill and vacuum preloading.

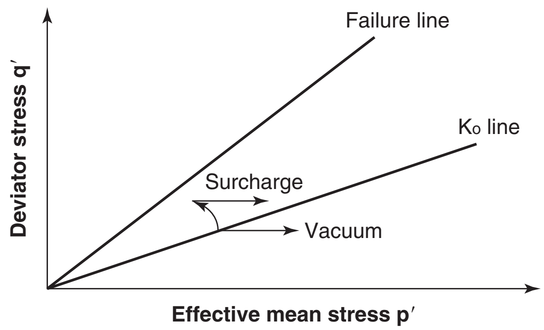


Figure 26.8 Stress path comparison between surcharge preloading and vacuum preloading. (After Chu et al. 2009.)

The construction sequence consists of constructing a 0.3 m thick sand blanket on the site, installing prefabricated vertical drains on a square grid (say, 1 m center to center), laying down a grid of geotextile-covered perforated pipes in the sand blanket to connect the PVDs to the vacuum pump, and covering the ground surface with a geomembrane to seal the soil volume. The vacuum pump is turned on and vacuum consolidation takes place. A variant of this process is shown in Figure 26.9.

The vacuum preloading method works well when the soil is soft, low permeability, and relatively homogeneous. If clean

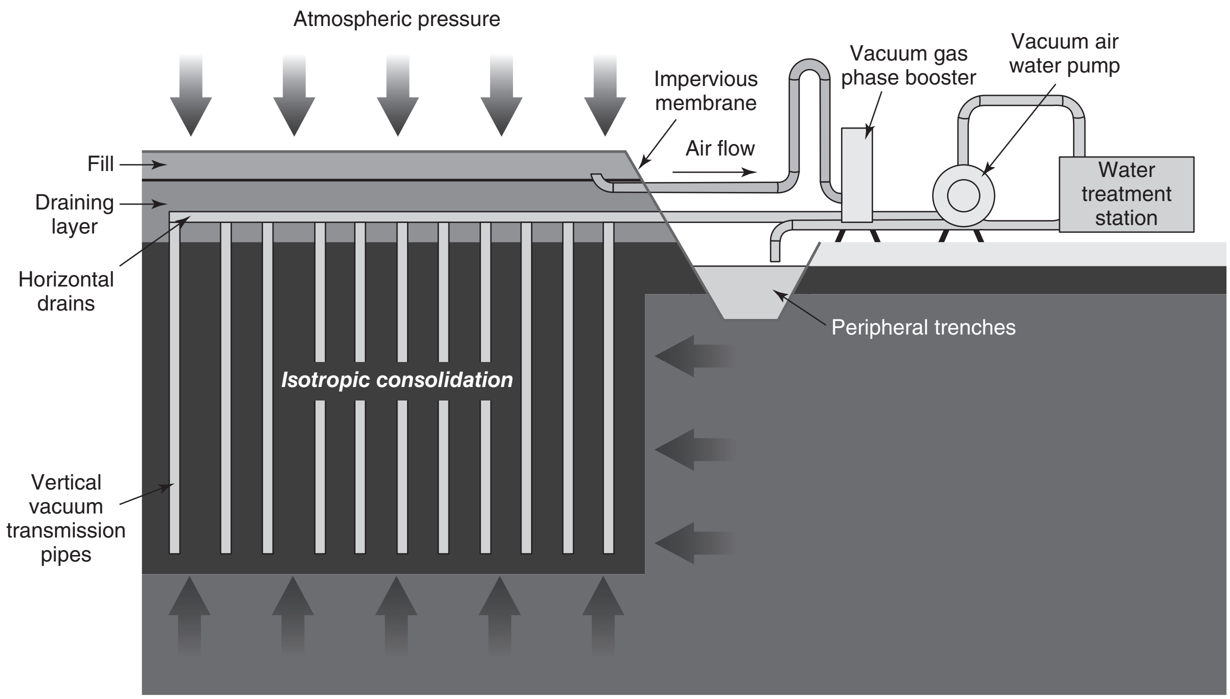


Figure 26.9 Menard vacuum consolidation. (Courtesy of Menard, Bridgeville, PA; www.menardusa.com)

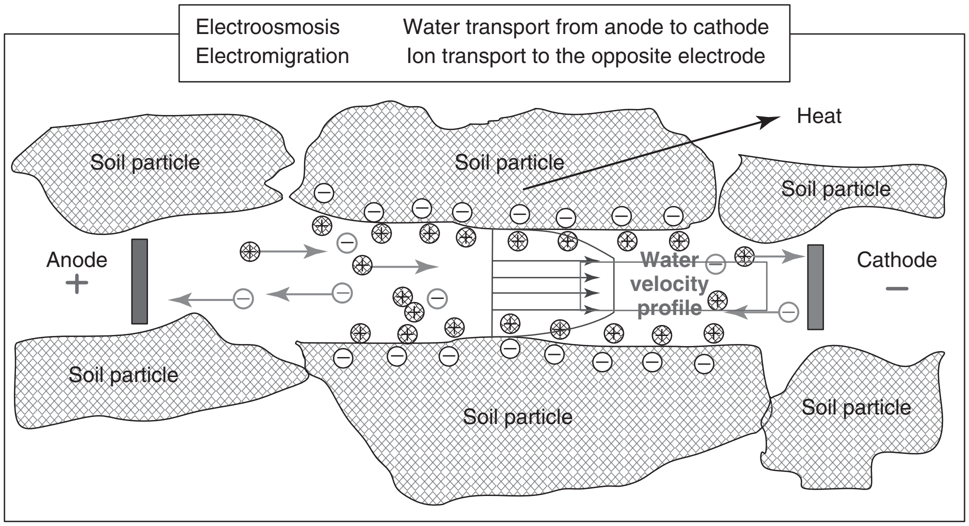


Figure 26.10 Electro-osmosis in clays. (Courtesy of C.J. Athmer—Terran Corporation.)

sand layers are interbedded in a deposit of soft clay, the efficiency of the process decreases unless cutoff walls can be installed first. Also, because vacuum preloading is isotropic, compression occurs in all direction equally and horizontal shortening takes place. This leads to vertical cracks in the soil mass.

26.3.5 Electro-osmosis

The *electro-osmosis* process was discovered in the early 1800s and applied to soils by Leo Cassagrande in the early 1940s. It is based on the fact that when a DC electrical current is established between two electrodes (e.g., steel bars) driven into fine-grained soil, the water flows from the anode (positive charge) to the cathode (negative charge) (Figure 26.10). The reason for this water movement is as follows. Clay particles are negatively charged and as such attract cations (positively charged) such as sodium, calcium and magnesium to their surfaces. When a DC current is established between two metal rods, the cations that line the surface of the clay particles start sliding toward the cathode by electrical attraction. The movement of this boundary layer of cations drags the bulk soil water with it. The water that accumulates at the cathode is drained away and the water content of the clay decreases, with an associated increase in strength and stiffness.

26.3.6 Ground Freezing

The technique of ground freezing (Figure 26.11) consists of freezing the soil by installing a network of steel pipes and circulating either brine water or liquid nitrogen. The temperature of circulating brine water is typically −20C; liquid nitrogen is much colder, at around −200C. Brine is much less expensive, but nitrogen takes a lot less time to freeze the ground. As a result, brine is used for large projects, whereas nitrogen may be economical when time is more important than cost savings. The advantage of ground freezing is that it is applicable to almost all soil conditions as long as the soil is saturated. Recall, however, that when water turns to ice, it expands by 10%. Applications include tunneling, retaining walls, cutoff walls, and contamination remediation.

Figure 26.11 Ground freezing. (Courtesy of British Drilling and Freezing Co. Ltd.)

26.3.7 Hydro-Blasting Compaction

The hydro-blasting compaction technique is particularly well suited to the treatment of collapsible soils. It consists of wetting the soil to induce collapse and then detonating explosives in sequence to shake the soil into a more compact arrangement.

26.4 SOIL IMPROVEMENT WITH REPLACEMENT

26.4.1 Stone Columns without Geosynthetic Sock

Stone columns, also called *aggregate columns* (Figure 26.12), are constructed by opening holes in the soil to be improved (say, 1 m diameter) down to a chosen depth (say, 10 m) and backfilling them with aggregates or crushed stones. Opening of the hole in which to place the stones is done by vibration or by jetting. In the vibration technique, a vibrating cylinder is used (section 26.2.3) and the stones are placed upon withdrawal and are compacted using the same vibrator. In the jetting technique, the hole is created by a probe inserted to the chosen depth and rotated out of the hole while jetting horizontally to enlarge the hole before the stones are placed. A third technique, called the *rammed aggregate pier method*, consists of opening a hole with an auger and compacting the stones in the open hole in 0.3 m thick lifts.

In all cases a stone column is placed in the soil to reinforce it vertically. This column can carry vertical compression load, but very little uplift load and horizontal load. It can also carry shear load, as required for the stabilization of unstable slopes. This latter case is handled as a slope stability problem. The rest of this section deals with the vertical compression capacity and settlement of stone columns.

The column can be considered as a large sample of gravel loaded in a manner similar to a triaxial test. Therefore, at failure of the column, the ratio between the vertical effective stress σ_1' and the horizontal effective stress σ_3' is given by:

$$\sigma_1' = K_p \sigma_3' \tag{26.11}$$

where K_p is the coefficient of passive earth pressure.

In this large-scale triaxial test, σ_3' is limited by the maximum horizontal pressure that the soil can resist. This is given by the effective stress limit pressure p_L' of the pressuremeter test. The value of p_L' can be obtained by performing a drained pressuremeter test (pressure steps lasting until the probe volume stabilizes) and assuming that the water stress u_w is equal to the hydrostatic pressure:

$$p_L' = p_L - u_w \tag{26.12}$$

Figure 26.12 Stone column construction. (Courtesy of Menard Bachy, Inc.)

Therefore, the drained ultimate load on the stone column is:

$$Q_u = K_p(p_L - u_w)A \tag{26.13}$$

where Q_u is the ultimate load on the stone column, p_L is the limit pressure from a drained pressuremeter test, u_w is the hydrostatic pressure at the PMT testing depth, and A is the cross-sectional area of the stone column. Of course, there is a beneficial effect that increases when the spacing between stone columns decreases; this observation makes Eq. 26.13 conservative.

The settlement can also be estimated using pressuremeter data. The horizontal relative expansion of the column is considered to be equal to the relative expansion of the pressuremeter for the same horizontal pressure:

$$\frac{\Delta B}{B} = \frac{\Delta R}{R} \tag{26.14}$$

where B and ΔB are the initial diameter and increase in diameter of the stone column respectively, and R and ΔR are the radius and increase in radius of the pressuremeter probe at a pressure corresponding to p_L divided by a chosen factor of safety against horizontal expansion failure. Therefore, ΔB can be obtained from Eq. 26.14. The volume involved in the barrel-like deformation shown in Figure 26.13 extends to a depth equal to about 2 times the diameter of the stone column (Hughes and Withers 1974). Thus, the initial volume involved in the deformation is:

$$V_o = 2B\frac{\pi B^2}{4} \tag{26.15}$$

If, during the deformation of the column under load, the volume of stone experiences a volume change ΔV, then the volume V of the deformed column under load will be:

$$V = V_o + \Delta V \tag{26.16}$$

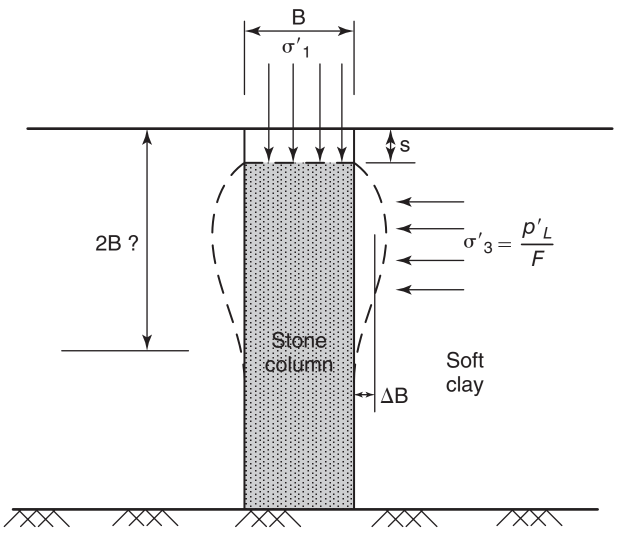


Figure 26.13 Expansion of a stone column under load.

The deformed volume V is also equal to:

$$V = (2B - s)\frac{\pi}{4}(B + \Delta B)^2 \qquad (26.17)$$

where s is the settlement of the stone column. This settlement s is then given by:

$$s = 2B\left(1 - \frac{\left(1 + \frac{\Delta V}{V_o}\right)}{\left(1 + \frac{\Delta B}{B}\right)^2}\right) \qquad (26.18)$$

The relative increase in stone column diameter $\Delta B/B$ is obtained from Eq. 26.14 using a ratio $\Delta R/R$ from a pressuremeter test at a pressure corresponding to p_L divided by a chosen factor of safety against horizontal expansion failure. The relative change in volume $\Delta V/V$ can be obtained from a triaxial test on the stone column material. The value of $\Delta V/V$ is the one that corresponds to a vertical stress σ_1' applied at the top of the stone column. Therefore, the settlement s corresponds to a top load Q equal to:

$$Q = \sigma_1' \frac{\pi B^2}{4} \qquad (26.19)$$

If ΔV is 0 and if $\Delta R/R$ is small, then Eq. 26.18 reduces to:

$$s = 4B\frac{\Delta R}{R} \qquad (26.20)$$

Another possible mode of failure is sliding along the sides of the column as a pile. The rules of design for piles can be used in this case, assuming that the failure will take place in the soft clay rather than the stone column at the vertical friction interface.

Besides strengthening the soft soil, stone columns act as large drains. When the surface is loaded, the water squeezes out of the soil horizontally (because the drainage length is shorter in that direction), drains into the stone column, and is collected at the surface. The design of stone columns as drains follows the same process as for prefabricated vertical drains (section 26.3.3).

26.4.2 Stone Columns with Geosynthetic Encasement

More recently, geosynthetic encasement, in the form of a large sock (Figure 26.14), has been used to increase the horizontal resistance and therefore vertical capacity of the stone column. Because improved horizontal drainage is also an attribute of stone columns, the geosynthetic used is a geotextile that provides a filter between the native soil (often soft clay) and the stone column material.

The critical factors for the encasement are the tensile capacity of the geotextile T_u (kN/m) and its modulus E (kN/m). The value of T_u ranges from 25 to 60 kN/m and that of E from 30 to 150 kN/m. The modulus E is defined as:

$$E = \frac{T}{\varepsilon} \qquad (26.21)$$

Figure 26.14 Stone column with geotextile encasement. (Courtesy of HUESKER Inc.)

where T is the force applied per meter of fabric and ε is the corresponding tensile strain.

Note that for geotextiles, the tensile strain at failure ε_f is very large, in the range of 25 to 70%. Therefore, it is likely that the soil would fail before the geotextile sock did. The failure mechanism may involve failure of the column aggregate, failure of the geotextile encasement, or failure of the soil laterally. Because of the large strains required for the geotextile to fail, this failure mechanism is not likely. The ultimate pressure that can be placed at the top of the encased stone column is given by:

Soil fails laterally $\quad p_{u1} = k_p\sigma_3' = k_p(p_L' + p_{geo}) \qquad (26.22)$

Geotextile fails in hoop tension

$$p_{u2} = k_p\sigma_3' = k_p\left(2G\frac{\Delta r}{r_o} + p_{geo\ f}\right) \qquad (26.23)$$

where p_{u1} is the ultimate pressure that can be placed at the top of the stone column if the soil fails first by reaching the soil effective stress horizontal limit pressure p_L', k_p is the coefficient of passive earth pressure of the soil, σ_3' is the horizontal stress generated by the combination of geotextile and soil, p_{geo} is the pressure contributed by the geotextile when stretched at $\Delta r/r_o$, $p_{geo\ f}$ is the pressure contributed by the geotextile at failure of the geotextile, G is the shear modulus of the soil outside the geotextile (soil being improved),

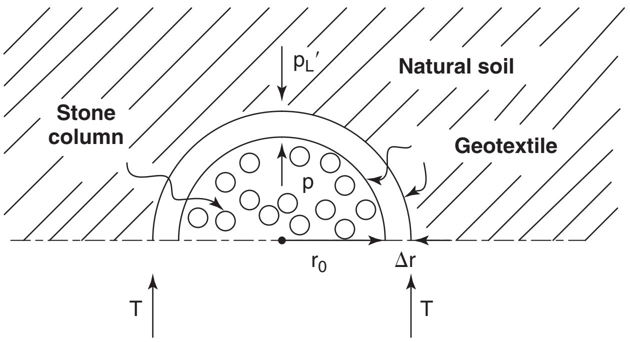


Figure 26.15 Pressure and tension in the geotextile encasement.

and $\Delta r/r_o$ is the relative increase in radius of the stone column. The expression $2G\Delta r/r_o$ is the pressure contributed by the soil outside the geotextile for a radial strain $\Delta r/r_o$.

Because the geotextile strain at failure is often very large, p_{u1} is likely to control. The pressure p_{geo} contributed by the geotextile at a relative increase in radius of the stone column equal to $\Delta r/r_o$ is given by (Figure 26.15):

$$p_{geo}2r = 2T = 2E\varepsilon = 2E\frac{\Delta r}{r_o} \quad \text{or} \quad p_{geo} = E\frac{\Delta r}{r_o^2} \tag{26.24}$$

Then the ultimate load on the encased stone column corresponding to this failure mechanism is:

$$Q_{u1} = k_p\left(p'_L + E\frac{\Delta r}{r_o^2}\right)\pi r_o^2 \tag{26.25}$$

The value of k_p is obtained from the friction angle of the stone column material, p'_L from a pressuremeter test in the natural soil, E from the geotextile material, r_o from the size of the stone column, and $\Delta r/r_o$ as 0.41 to correspond with the strain at failure for the limit pressure. Note that the product $E\ \Delta r/r_o$ cannot be larger than the tensile capacity T_u of the geotextile.

The settlement calculations become rather cumbersome in close form and are best handled by numerical simulations starting with elasticity. Alexiew et al. (2003) and Raithel et al. (2005) proposed simplified method for hand calculations.

26.4.3 Dynamic Replacement

Dynamic replacement (DR) starts by placing a blanket of aggregates on top of the soil to be improved. Then a dynamic compaction (DC) operation is performed, creating craters that are filled with aggregates to form a plug. More pounding takes place on top of the plug at the same locations; the craters deepen and more aggregates are placed in the open hole. The process is repeated until the crater decreases in depth. In this fashion a column of compacted aggregates is formed in place (Figure 26.16). The same range of weight, drop height, and pounder diameter are used for both DC and DR. If the energy used is high (200 to 400 kJ/m^3) and the soil is softer (PMT limit pressure 100 to 400 kPa), then the craters are deep, DR takes place, and the degree of improvement is high. In contrast, if the energy is lower (50 to 250 kJ/m3) and the soil is stronger (PMT limit pressure 250 to 700 kPa), then the craters are limited in depth, DC take place, and the degree of improvement is lower.

26.5 SOIL IMPROVEMENT WITH GROUTING AND ADMIXTURES

You might have heard the words *grout, concrete, cement*, and *mortar*: what are they, and what is the difference?

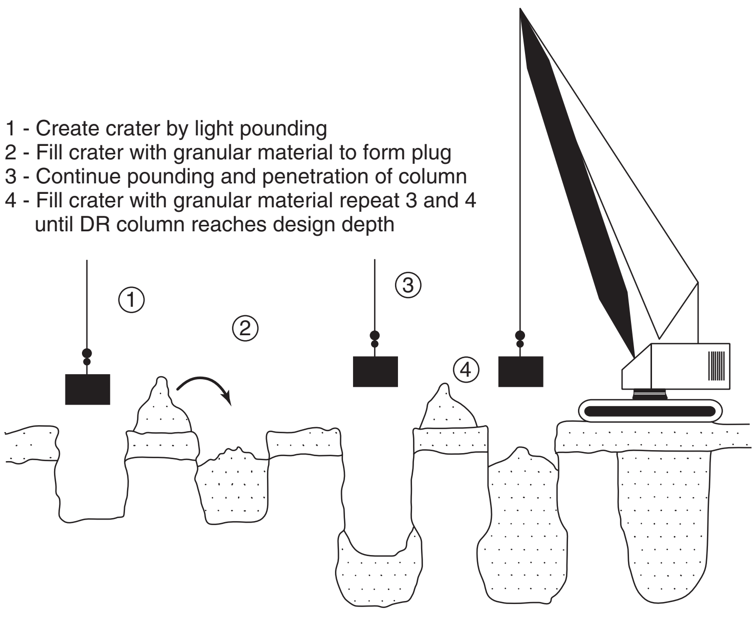


Figure 26.16 Dynamic replacement method (After Chu et al. 2009).

Cement is made of calcium and silicon. If you want to make cement in your kitchen, you mix powdered limestone (calcium carbonate, $CaCO_3$) and powdered clay (mostly silica, SiO_2) and heat it to 1450°C,; you will get a hard piece of rock out of the oven. (Note that the oven in your kitchen is very unlikely to be able to reach such high temperatures.) If you then grind that piece of rock into a very fine powder, you will have a crude cement. When you add water to that very dry cement powder, an exothermic reaction (generates heat) called *hydration* takes place and produces calcium silicate hydrate, which is the main source of cement strength. Cement is the binder in concrete, mortar, and grout. *Concrete* is the combination of cement, water, sand, gravel, and even larger aggregates. *Mortar* and *grout* are combinations of cement, water, and fine sand. The difference between mortar and grout is that typically grout will be more fluid than mortar. Sometimes grout is simply cement and water.

Different grouting techniques are used depending on the type of soil to be improved (Warner 2004). For gravels and coarse sands, the grout is injected by gravity or under pressure and fills the soil voids; the smaller the D_{50} of the soil, the finer, the more fluid, and the less viscous the grout has to be. These techniques include particulate grouting and chemical grouting. For fine sands and fine-grained soils, the grout is placed in a hole made in the soil to be improved. These techniques include jet grouting, compaction grouting, and compensation grouting. Also, for fine-grained soils, the soil can be mixed with grout that acts as a drilling fluid; this is *soil mixing*. Figure 26.17 shows the range of applicability of various grouting techniques.

26.5.1 Particulate Grouting

Particulate grouting refers to grouting coarse-grained soils by injecting the grout under gravity or under pressure into the soil voids. It also refers to grouting fissures in rocks and cavities such as sinkholes. Particulate grouting consists of opening a borehole down to the desired depth, sealing it, and then injecting the grout. The spacing between boreholes is in the range of 1 to 2 m and the hydraulic conductivity of the soil for which this technique is applicable is 10^{-2} to 10^{-5} m/s. The tube a manchettes (TAM) technique can be used to inject the grout into the soil under pressure. The TAM consists of a casing with holes at regular intervals (say, 0.5 m) covered by rubber sleeves. Two packers inside the TAM casing are inflated, one above the holes and one below; then the grout can be injected through that hole to force the rubber sleeve to lift off and allow the grout to flow into the adjacent soil under pressure. The pumping rate for particulate grouting can vary from 0.1 to 25 m^3/hr under a pressure of 0.5 to 10 MPa.

The groutability of soils is often evaluated through a ratio N of the soil grain size to the grout grain size. For example:

$$N_1 = \frac{D_{10(soil)}}{D_{65(grout)}} \quad \text{or} \quad N_2 = \frac{D_{10(soil)}}{D_{95(grout)}} \tag{26.26}$$

where $D_{10(soil)}$ is the grain size of the soil corresponding to 10% fines, and $D_{65(grout)}$ and $D_{90(grout)}$ are the grain size of the grout corresponding to 65 and 90% fines respectively.

Mitchell and Katti (1981) state that grouting is feasible if $N_1 > 24$ and not feasible if $N_1 < 11$. Karol (2003) states that grouting is feasible if $N_2 > 11$ and not feasible if $N_2 < 6$. Groutability also depends on how fluid the grout is and what injection pressure is applied. Akbulut and Saglamer (2002) proposed a more complete expression that reflects the influence of these parameters:

$$N_3 = \frac{D_{10(soil)}}{D_{90(grout)}} + k_1 \frac{w/c}{FC} + k_2 \frac{P}{D_r} \tag{26.27}$$

where k_1 and k_2 are soil-specific factors (0.5 and 0.01 for the soil tested by Akbulut and Saglamer), w/c is the water-to-cement ratio of the grout, FC is the fine content, P is the grout pressure in kPa, and D_r is the soil relative density. The soil is considered groutable if N_3 is larger than 26.

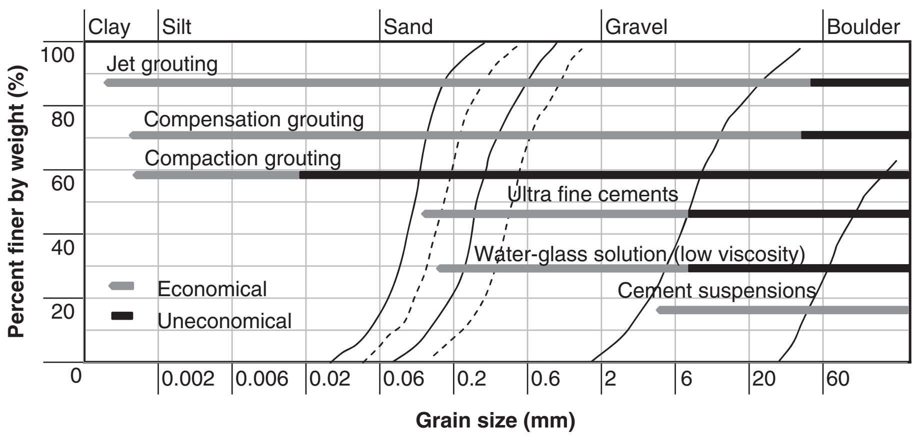


Figure 26.17 Range of application of grouting techniques (After Keller 2012; www.keller grundbau.com/download/pdf/en/Keller_66–01E.pdf).

26.5.2 Chemical Grouting

Chemical grouting makes use of any grout that is a pure solution with no particles in suspension. Because it does not have any solids in suspension, it can penetrate finer soils. Whereas the groutability of soils by particulate grouts depends on the grain size of the solids in the grout, the groutability of chemical grouts depends on their viscosity. Chemical grout can be used in soils as fine as coarse silt.

26.5.3 Jet Grouting

Particulate and chemical grouts permeate the soil and fill the voids with grout. These techniques apply mostly to coarse-grained soils. For fine-grained soils, it is not possible for the grout to penetrate the voids, because they are too small. Instead, the approach consists of creating columns of grout in place. This is done by jet grouting, or compaction grouting, or compensation grouting, or soil mixing. Note that these techniques are also applicable to coarse-grained soils (Figure 26.17).

Jet grouting consists of drilling a borehole down to the desired depth. The drill bit has a diameter in the range of 100 to 150 mm. Once the required depth is reached, a horizontal high-pressure jet (~20 MPa) is generated to erode the soil laterally. The rod is withdrawn while rotating (Figure 26.18). This erosion process generates a larger-diameter hole (1 to 1.5 m) that is then filled with grout.

26.5.4 Compaction Grouting

Compaction grouting (Figure 26.19; Al-Alusi 1997) consists of drilling a hole with a small casing to the depth where grouting is to start, and then injecting very stiff grout with 25 mm slump or less (decrease in height in a standard cone test) under 3 to 7 MPa pressure while withdrawing the grout casing. The grout injection is performed at discrete locations and forms bulbs of grout that are 0.3 to 0.6 m thick. The grout does not penetrate the soil voids, but instead displaces and densifies the soil around the bulb. A sudden drop in pressure often indicates soil fracture. The spacing between grouting holes is in the range of 2.5 to 3.5 m center to center.

26.5.5 Compensation Grouting

Compensation grouting is used to minimize the amount of soil deformation potentially created by excavation and tunneling. It consists of injecting a volume of grout that compensates for the volume of soil displaced so that the adjacent ground surface or buildings do not deflect excessively. The grout can be injected by intrusion grouting, fracture grouting, or compaction grouting. The method is used in many different types

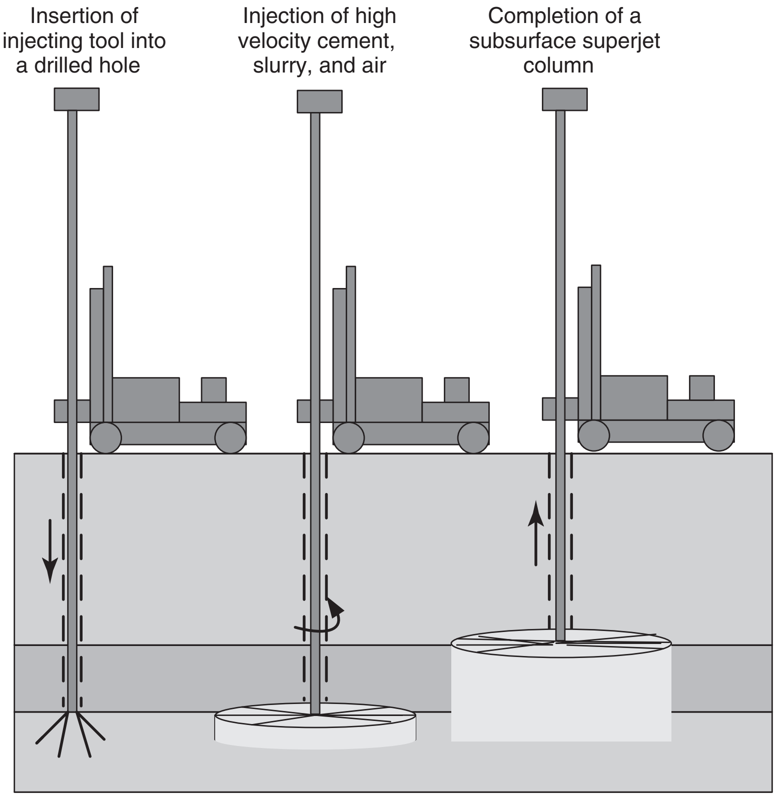


Figure 26.18 Jet grouting (After Hayward Baker, Inc.).

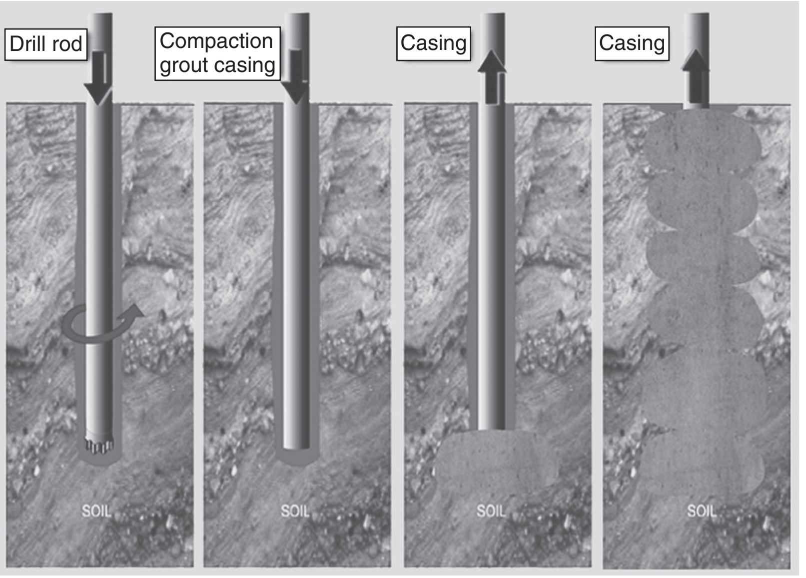


Figure 26.19 Compaction grouting. (Courtesy of Arizona Repair Masons Inc.)

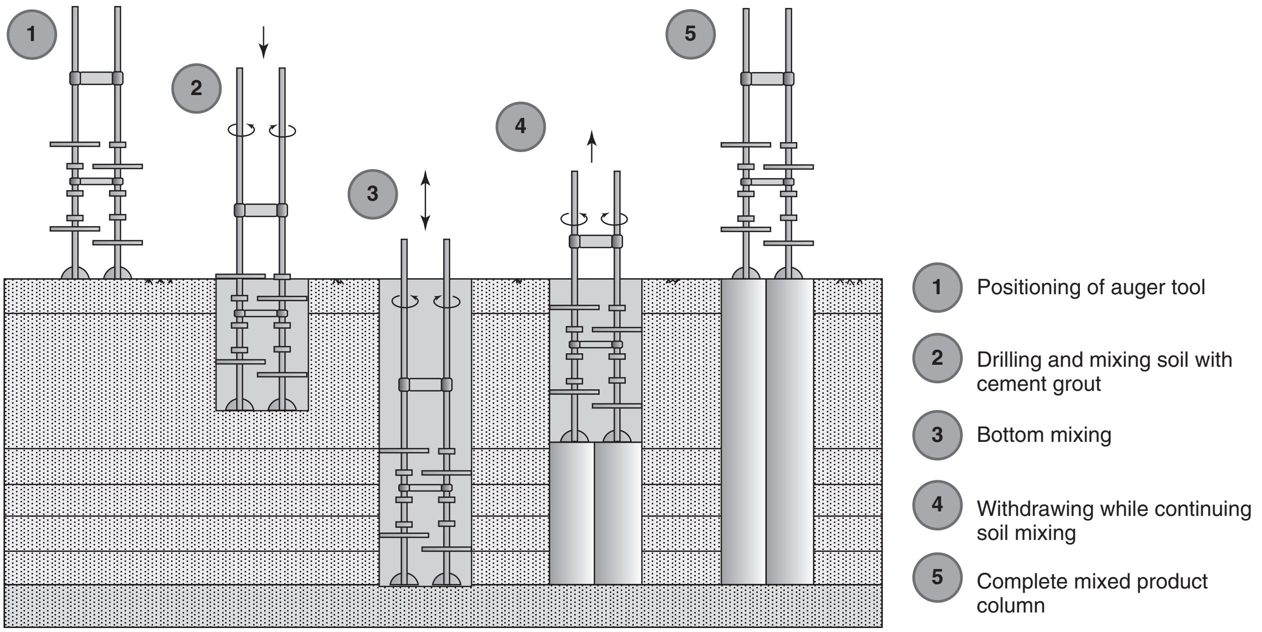


Figure 26.20 Example of SCM excavation support construction sequence. (Courtesy of JAFEC USA, Inc. Geotechnical Constructors)

of soils, but mostly in fine-grained soils, although difficulties have been encountered in soft clays (Chu et al. 2009).

26.5.6 Mixing Method

The mixing method consists of mixing soil with grout in place. The grout serves as the slurry for the drilling process and the soil-grout mixture creates a strengthened column in situ. The technique is called *deep soil mixing* (DSM) or *deep cement mixing* (DCM) or *soil cement mixing* (SCM). The drilling tool is usually a paddle auger (Figure 26.20) about 1 m in diameter; several side-by-side augers can be used at one time. Examples of construction with SCM include walls for deep excavation in soft clays, flow barriers, or simply forming a block of strengthened soil mass. The ratio of grout to soil varies from 0.15 to 0.4. Soils usually have compressive strengths less than 200 kPa and concrete more than 20,000 kPa; in SCM the soil-cement mixtures have unconfined compressive strength in the 2000 kPa range. The modulus of deformation of SCM varies in the range of 100 to 1000 MPa and can be estimated by the following equation (Briaud and Rutherford 2010):

$$E_{Soil\ Cement}\ (\text{kPa}) = 12{,}900(f_c'\ (\text{kPa}))^{0.41} \qquad (26.28)$$

26.5.7 Lime Treatment

If you want to make lime, you take a piece of natural limestone rock ($CaCO_3$); heat it to about 1000°C, which drives the carbon (CO_2) out of the limestone, and then grind the leftover piece of rock. You will have a white powder called *lime* or *calcium oxide* (CaO). If you then mix this white powder with a wet clay, it will hydrate, reabsorb carbon dioxide, and turn back into limestone. The difference between cement and lime is that lime does not strengthen as rapidly as cement; also, it is not as strong and more brittle than cement. The strengthening of the lime-soil mixture is accompanied by a decrease in water content of the clay, an increase in pH (more alkaline), an increase in plastic limit, a decrease in plasticity index, and a decrease in shrink-swell potential. The lime affects the electrostatic field around the clay particles, which tend to flocculate and assume a more granular structure. The typical amount of lime added to a clay ranges between 2 and 8%. The design of the mix and the impact on the soil properties are given in Little (1999). The clay-lime mixture has unconfined compression strengths between 700 and 1400 kPa and moduli between 200 and 3000 MPa. Lime treatment is often used to stabilize pavement foundation layers, and works best when the soil has at least 25% passing the 75 micron sieve and a plasticity index (PI) of at least 10. In the field, the lime is mixed with the surface soil and hydrated (Figure 26.21).

There is one case in which using lime can be very counterproductive: This is the case where the soil to be stabilized contains a certain amount of sulfate in the form of gypsum ($CaSO_4\ 2H_2O$). The addition of lime (CaO) and water (H_2O) to this type of soil will form ettringite ($Ca_6Al_2(SO_4)_3(OH)_{12}$

After applying lime slurry to prepared soil, the machine is run in reverse to ensure thorough mixing to the specified depth.

Figure 26.21 Lime stabilization of pavement layers. (Photo by James Cowlin/Asphalt Busters, Phoenix, AZ.)

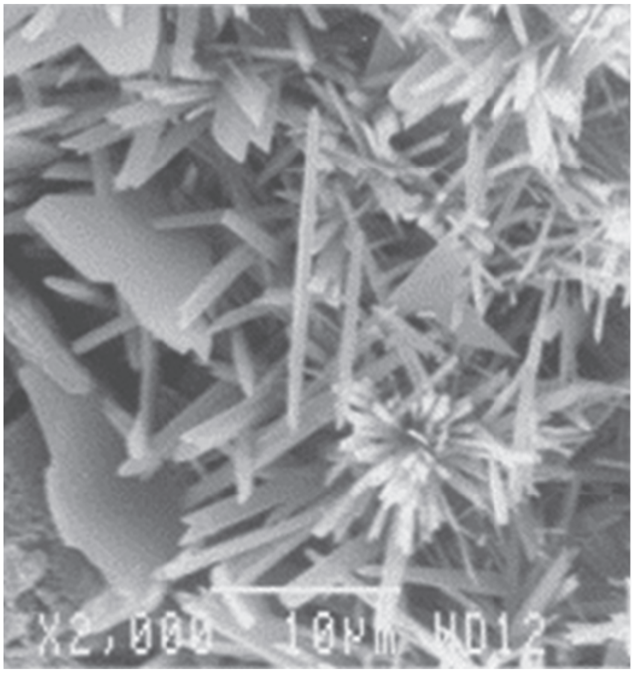


Figure 26.22 Ettringite crystals. (Courtesy of www.sciencedirect.com/science/article/pii/S0008884698001379)

$26H_2O$), which is a highly expansive mineral. Ettringite crystals are needle like (Figure 26.22) and when mixed with water can swell to 250% of their initial height and destroy pavements. If the total soluble sulfate level is greater than about 0.3% in a 10-to-1 water-to-soil solution, additional precautions to guard against this sulfate reactions, such as swell tests, may be warranted (Little 1999).

26.5.8 Microbial Methods

Certain naturally occurring bacteria are able to generate material that can either plug the soil voids (bio-plugging) or cement particles together (bio-cementation). Water-insoluble microbial slime is produced by facultative anaerobic and microaerophilic bacteria to plug the soil voids. Bio-plugging can decrease the hydraulic conductivity of the soil by a factor of 2 (Ng et al. 2012). Calcite is produced by ureolytic bacteria that precipitate calcium carbonate.

Bio-cementing increases the shear strength of the soil by cementing the particles together. This process is called microbial-induced calcite precipitation (MICP) and works best with sand particles. It takes place when the urease enzyme produced by bacteria such as *Bacillus megaterium* decomposes urea by hydrolysis and produces ammonium. In turn, ammonium increases the pH and starts the precipitation of calcium carbonate. Calcium carbonate is the glue that cements the soil grains together (Figures 26.23 and 26.24) and can increase the shear strength of the sand by a factor of 2 (Ng et al. 2012).

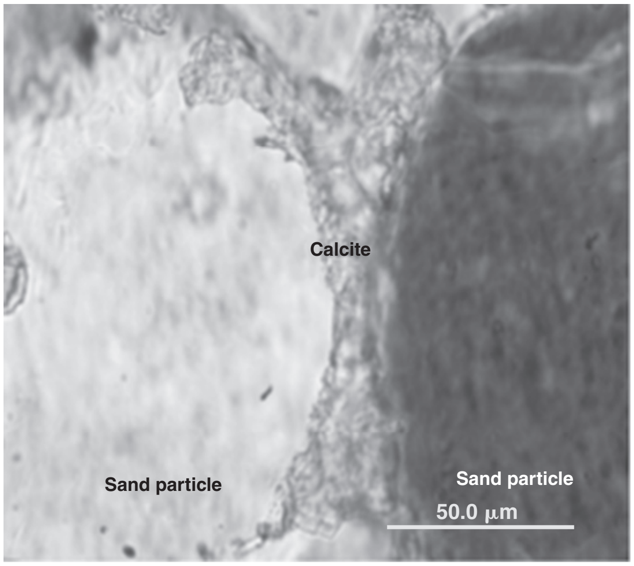


Figure 26.23 Light microscopic image of calcite crystals, produced by ureolytic bacteria, cementing two sand particles. (Courtesy of Salwa Al-Thawadi.)

26.6 SOIL IMPROVEMENT WITH INCLUSIONS

26.6.1 Mechanically or Geosynthetically Stabilized Earth

Mechanically stabilized earth (MSE) walls are covered in section 21.10. Geosynthetically stabilized earth (GSE) walls are covered in section 25.6.2.

26.6.2 Ground Anchors and Soil Nails

Ground anchor walls are covered in section 21.12 and soil nail walls are covered in section 21.13.

26.6.3 Geosynthetic Mat and Column-Supported Embankment

Geosynthetic mat and column-supported embankments (GMCSs) (Figure 26.25) are increasingly being used as a way to rapidly construct or widen embankments on soft soils. The construction proceeds by first constructing the columns to the required depth, and preferably to a strong layer; then covering them with a bridging layer made of interbedded select fill and geosynthetic layers (say, 1 m thick); and then completing the embankment to the design height. The design process proceeds as follows (Smith 2005; Schaefer 2013):

1. Investigate the site to collect the properties of the natural soil.
2. Choose the depth and spacing of the columns. Identify the repeatable shape in plan view of the group of columns called the *unit cell*. The depth should be chosen such that the columns reach a strong layer. The spacing s should be smaller than the following values:

$$\begin{aligned} s &\leq 0.67H + a \\ s &\leq 1.23H - 1.2a \\ s &\leq a + 3 \text{ meters} \end{aligned} \tag{26.29}$$

where H is he height of the embankment, a is the side of the individual square cap on top of the column. If there is no cap, a is taken as 0.89 times the diameter of the column (equivalent areas).

Center-to-center spacings of between 2 and 5 column diameters are common. The conditions placed on the spacing (Eq. 26.29) are set to ensure that proper arching will develop in the embankment through the bridging

(a) (b)

Figure 26.24 Microbial-induced calcite precipitation: (a) Bacteria and calcium chloride. (b) Brick-like product. (Courtesy of Ginger Krieg Dosier, bioMASON Inc.)

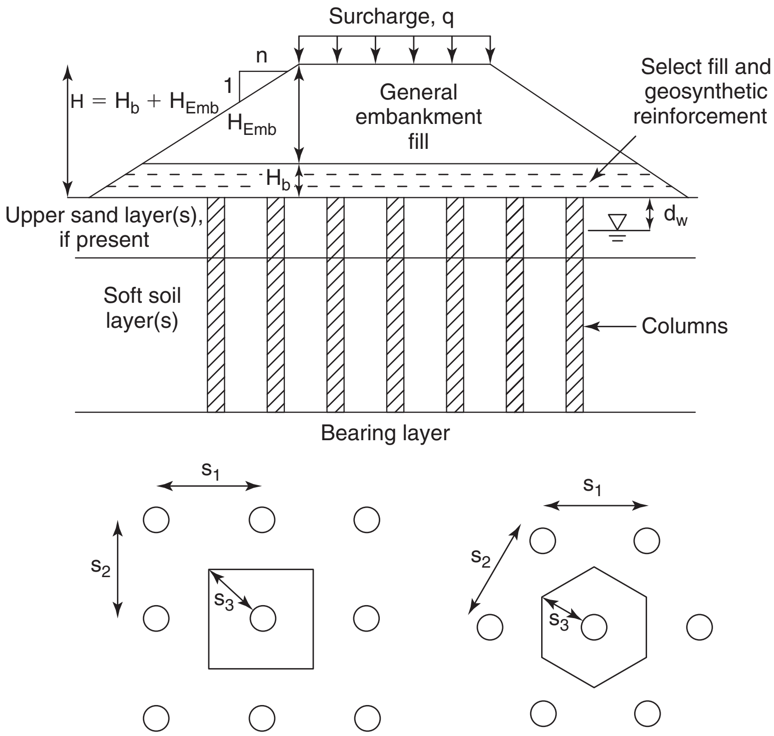


Figure 26.25 Geosynthetic reinforced column supported embankment. (Courtesy of Professor Vernon Schaefer, vern@iastate.edu)

layer to transfer all the embankment weight to the columns. Arching is also ensured by selecting a material for the bridging layer that satisfies a number of criteria (Schaefer 2013).

3. Determine the column load Q_{col} knowing the height of the embankment:

$$Q_{col} = (\gamma H + q)A \tag{26.30}$$

where γ is the unit weight of the embankment fill, H is the height of the embankment, q is the traffic surcharge, and A is the tributary area of the column or unit cell (Figure 26.25).

4. Design the piles to safely carry Q_{col}. See sections 18.4 and 18.5.
5. Calculate the tension load in the geosynthetic layer. This tension load has two components: the tension load T_1 due to the vertical load transferred from the embankment to the columns through the bridging layer, and the tension load T_2 due to the tendency of the embankment to spread laterally. Filz et al. (2012) recommend the following expression for calculating the value of T_1:

$$6{T_1}^3 - (6T_1 - E_{GS})\left(\frac{\sigma_{net} A_{soil}}{p}\right) = 0 \tag{26.31}$$

where T_1 is the tension load per unit length of embankment in the geosynthetic due to the embankment vertical load, E_{GS} is the modulus of the geosynthetic layers (kN/m), σ_{net} is the difference between the soil pressure above the geosynthetic and below the geosynthetic, A_{soil} is the area within the unit cell underlain by soil, and p is the column or pile cap perimeter.

The tension load T_2 is obtained from:

$$T_2 = \frac{1}{2} K_a \gamma H^2 + q K_a H \tag{26.32}$$

where T_2 is the tension load per unit length of embankment in the geosynthetic due to the embankment tendency to spread laterally, K_a is the coefficient of active earth pressure of the embankment soil, H is the embankment height, and q is the traffic surcharge.

6. Select a suitable geosynthetic. Geogrids are most commonly used for this application. The geosynthetic has two strengths that must be checked: the creep-limited strength at 5% strain and the allowable tensile strength (see section 25.3.3).
7. Calculate the embedment length L_e of the geosynthetic layer:

$$L_e \geq \frac{(T_1 + T_2)F}{\gamma H(\tan\delta_1 + \tan\delta_2)} \tag{26.33}$$

where F is the factor of safety, γ is the unit weight of the embankment soil, H is the height of the embankment, and $\tan\delta_1$ and $\tan\delta_2$ are the coefficients of friction between the geosynthetic and the soil above and below in the bridging layer.

8. Calculate the total settlement s of the embankment, which includes the compression of the embankment soil under its own weight, the compression of the columns under load, and the settlement of the group of columns (see section 18.5). If the settlement is excessive, the spacing between columns can be reduced and the columns can be lengthened.
9. The lateral extent of the group of columns should be decided by stability analysis of the embankment slope reinforced by the columns (see section 19.14). A factor of safety of 1.3 to 1.5 is common.

26.7 SELECTION OF SOIL IMPROVEMENT METHOD

Considering how many different methods exist for soil improvement, it is important to have a tool that can optimize the choice of method for the given situation. The factors to be considered include the soil type; the fine content and size; the soil strength and compressibility; the area and depth of treatment; the proposed structure; the settlement criteria; the availability of skills, equipment, and materials; and the cost of the possible techniques. Sadek and Khouri (2000) proposed a software product called Soil and Site Improvement Guide to optimize the choice. More recently, Schaefer (2013) optimized the decision process through freeware available at www.geotechtools.org.

PROBLEMS

26.1 Three soils have the following CPT characteristics. Can they be vibrocompacted?

Soil	Point Resistance	Friction Ratio
Soil 1	10 MPa	1.2%
Soil 2	8 MPa	0.5%
Soil 3	15 MPa	2%

26.2 An embankment was built as shown in Figure 26.1s.
  a. What is the maximum settlement of the embankment?
  b. How much time is required for 90% of that settlement to occur?
  c. How much surcharge is required to get the maximum settlement in 6 months? (Assume that the stress increase in the clay layer is equal to the stress at the bottom of the embankment.)

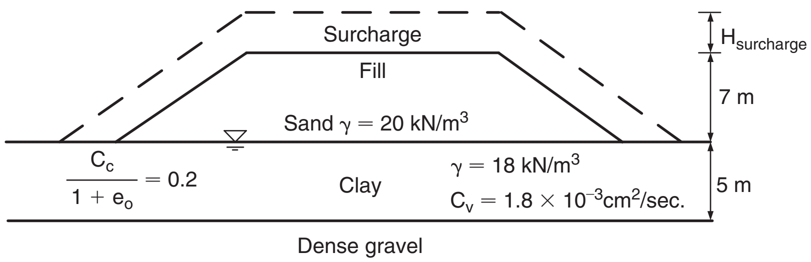


Figure 26.1s Highway embankment for preloading problem.

26.3 A highway embankment is to be built on a soft clay layer. What is the spacing of prefabricated vertical drains necessary to obtain 90% consolidation in 12 months? The PVDs are 100 mm wide and 4 mm thick, and are constructed on a square grid. The soil data are shown in Figure 26.2s.

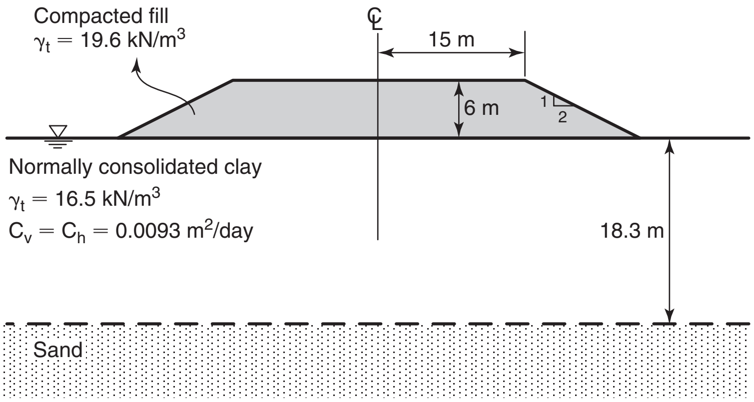


Figure 26.2s Highway embankment for prefabricated vertical drain problem.

26.4 A docking facility needs to have a 5 m high fill built on top of a 10 m thick soft silt layer underlain by dense sand. The groundwater level is at the ground surface. Stone columns 1 m in diameter and 10 m long are built. Long-term drained pressuremeter tests are performed and a conservative value of the limit pressure and modulus are 150 kPa and 1700 kPa respectively. The friction angle of the gravel used for the columns is 38°. Calculate the load that can be safely carried by one column and the settlement of the top of the column under that load. Assume that no volume change takes place in the column.

26.5 Repeat problem 26.4 but this time the stone columns are encased in a geotextile with a stiffness E equal to 150 kN/m and a tensile strength of 60 kN/m.

26.6 How would you make cement? How do you make lime? What is the difference between cement and lime?

26.7 A soil has a D_{10} equal to 2 mm. The grout used to strengthen it has a D_{65} of 60 μm and a D_{95} of 130 μm. Can particulate grouting be successful?

26.8 A geosynthetic mat and column-supported embankment (GMCS) is used to build an embankment on soft clay. The embankment is 7 m high, built with a fill with a compacted unit weight of 20 kN/m^3 and a friction angle of 32°. The columns are 1 m in diameter with no pile cap and are placed on a square 2 m center-to-center grid. The bridging layer is 1 m thick with two layers of geosynthetic. Calculate the load per column, the tension in the geosynthetic layers due to spanning across the columns (T_1), and the tension due to lateral spreading (T_2). Assume that the net difference in stress on either side of the geosynthetic layer is 80% of the pressure under the embankment and that the geosynthetic has a modulus equal to 60 kN/m.

Problems and Solutions

Problem 26.1

Three soils have the following CPT characteristics. Can they be vibrocompacted?

Soil	Point Resistance	Friction Ratio
Soil 1	10 MPa	1.2%
Soil 2	8 MPa	0.5%
Soil 3	15 MPa	2%

Solution 26.1

According to Massarsch guidelines, and Figure 26.2:

- Soil 1—Marginal
- Soil 2—Yes
- Soil 3—No

Problem 26.2

An embankment was built as shown in Figure 26.1s.

a. What is the maximum settlement of the embankment?
b. How much time is required for 90% of that settlement to occur?
c. How much surcharge is required to get the maximum settlement in 6 months? (Assume that the stress increase in the clay layer is equal to the stress at the bottom of the embankment.)

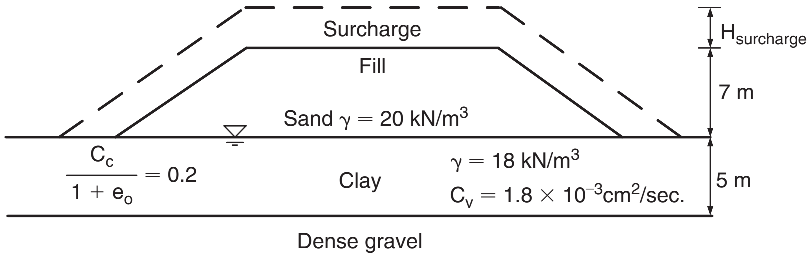


Figure 26.1s Highway embankment for preloading problem.

Solution 26.2

a. Maximum Settlement of the Embankment

$$S_{\max} = H\frac{C_c}{1+e_o}\log\frac{\sigma'_{ov} + \Delta\sigma'}{\sigma'_{ov}}$$

$$\sigma'_{ov} = 2.5 \times 18 - 2.5 \times 9.81 = 20.48 \text{ kPa}$$

$$\Delta\sigma' = 20 \times 7 = 140 \text{ kPa}$$

$$\therefore S_{\max} = 5 \times 0.2 \times \log\frac{20.48 + 140}{20.48} = 0.89 \text{ m}$$

b. Time Required for 90% Settlement

$$T_v = C_v\frac{t}{H_{dr}^2} \qquad \therefore t = T_v\frac{H_{dr}^2}{C_v}$$

when $U = 90\%$, $T_v = 0.848$

$$\therefore t = 0.848\frac{2.5^2}{1.8 \times 10^{-3} \times 10^{-4}} = 2.94 \times 10^7 \text{ sec.} = 341 \text{ days}$$

c. Surcharge Needed to Reach Maximum Settlement in 6 Months

The surcharge needed to reach the maximum settlement in 6 months is calculated as follows:

$$T_v = 1.8 \times 10^{-7} \times \frac{6 \times 30 \times 24 \times 3600}{2.5^2} = 0.448$$

When $T_v = 0.448$, $U = 73.2\%$

$$U = \frac{S_{6\ months\ \text{(fill+surcharge)}}}{S_{\max(fill+surcharge)}}$$

Because we want $S_{6\text{ months}}$ (fill + surcharge) $= S_{\max}$(fill), then:

$$U = \frac{S_{\max\ (fill)}}{S_{\max(fill+surcharge)}}$$

$$S_{\max}(F+S) = \frac{0.89}{0.732} = H_o\frac{C_c}{1+e_o}\log\frac{\sigma'_{ov} + \Delta\sigma'}{\sigma'_{ov}} = 5 \times 0.2 \times \log\left(\frac{20.48 + \Delta\sigma}{20.48}\right)$$

$$1.216 = \log\left(\frac{20.48 + \Delta\sigma}{20.48}\right)$$

$$\Delta\sigma = 316.3 \text{ kPa} = \Delta\sigma(F) + \Delta\sigma(S)$$

$$316.26 \text{ kPa} = 20 \times 7 + 20 \times h$$

$$h = 8.81 \text{ m}$$

Problem 26.3

A highway embankment is to be built on a soft clay layer. What is the spacing of prefabricated vertical drains necessary to obtain 90% consolidation in 12 months? The PVDs are 100 mm wide and 4 mm thick, and are constructed on a square grid. The soil data are shown in Figure 26.2s.

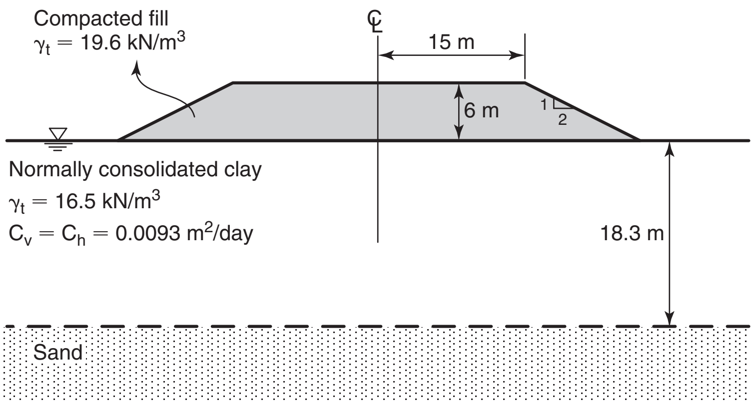


Figure 26.2s Highway embankment for prefabricated vertical drain problem.

Solution 26.3

The equation that gives the spacing is:

$$t = \frac{d_w^2}{8c_h}\left(Ln\left(\frac{d_w}{d_e}\right) - 0.75 + F_s\right)Ln\left(\frac{1}{1 - U_h}\right)$$

where d_e is the equivalent diameter of the PVD defined as $2(\text{a} + \text{b})/\pi$, a and b are the width and thickness of the PVD, c_h is the horizontal coefficient of consolidation, d_w is the well influence diameter (taken as 1.05 s for an equilateral triangle spacing pattern and 1.13 s for a square spacing pattern) where s is the spacing between PVDs, and F_s is a soil disturbance factor (taken as 2 for highly plastic sensitive soils but taken as zero if c_h has been conservatively estimated or accurately measured). Using the parameters given in the problem, we get:

$$12 \times 30 = \frac{(1.13s)^2}{8 \times 0.0093}\left(Ln\left(\frac{1.13s}{2\,(0.1 + 0.004)\,/\pi}\right) - 0.75 + 1\right)Ln\left(\frac{1}{1 - 0.9}\right)$$

Note that an average value of $F_s = 1$ is used in this case:

$$360 = \frac{1.277s^2}{0.0744}(Ln(17.07s) - 1.75) \times 2.303$$

$$9.108 = s^2(Ln(17.07s) - 1.75)$$

This equation is solved by trial and error and gives a center-to-center spacing of $s = 2.2$ m.

Problem 26.4

A docking facility needs to have a 5 m high fill built on top of a 10 m thick soft silt layer underlain by dense sand. The groundwater level is at the ground surface. Stone columns 1 m in diameter and 10 m long are built. Long-term drained pressuremeter tests are performed and a conservative value of the limit pressure and modulus are 150 kPa and 1700 kPa respectively. The friction angle of the gravel used for the columns is 38°. Calculate the load that can be safely carried by one column and the settlement of the top of the column under that load. Assume that no volume change takes place in the column.

Solution 26.4

$$Q_u = K_p(p_L - u_w)A$$

$$K_p = \frac{1+\sin 38}{1-\sin 38} = 4.2$$

$$Q_u = 4.2 \times (150 - 5 \times 9.8) \times 0.5^2\pi = 333 \text{ kN}$$

$$Q_{allowable} = \frac{Q_u}{F.S.} = \frac{333}{2} = 166.6 \text{ kN}$$

$$s = 4B\frac{\Delta R}{R}$$

$$\text{E} = 1700 \text{ kPa and } v = 0.35$$

$$\frac{\Delta R}{R} = \frac{P}{2G} = \frac{1+v}{E}\frac{Pl}{F.S.}$$

$$\frac{\Delta R}{R} = \frac{1.35}{1700}\frac{150}{2} = 0.06$$

$$s = 4 \times 0.06 = 0.24 \text{ m}$$

Problem 26.5

Repeat problem 26.4 but this time the stone columns are encased in a geotextile with a stiffness E equal to 150 kN/m and a tensile strength of 60 kN/m.

Solution 26.5

Figure 26.3s illustrates this problem.

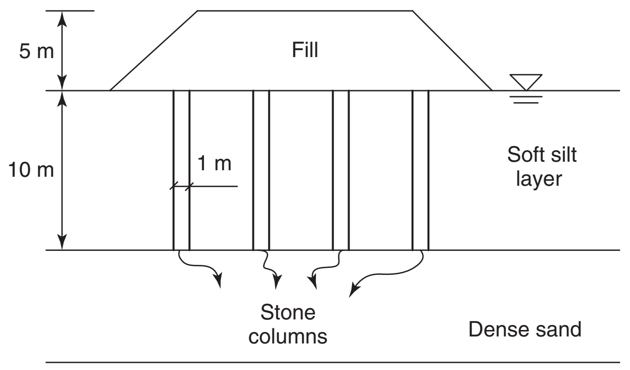


Figure 26.3s Illustration of the soil profile.

Failure mechanism 1: Soil fails laterally

The ultimate pressure the stone column can carry is calculated as:

$$p_{u1} = k_p(p'_L + p_{geo})$$

where k_p is the passive earth pressure coefficient of the soil, p'_L is the drained pressuremeter limit pressure, and p_{geo} is the lateral confinement pressure generated by the geotextile at the strain corresponding to failure of the soil.

$$k_p = \frac{1+\sin\varphi'}{1-\sin\varphi'} = \frac{1+\sin 38}{1-\sin 38} = 4.2$$

$$p'_L = p_L - u_w = 150 - 5 \times 9.81 = 101 \text{ kPa}$$

The limit pressure is associated with a radial strain or hoop strain equal to 41%. We use this strain to calculate p_{geo}. Therefore, the deformation of the geotextile at failure is:

$$\Delta r = r_o \times \varepsilon = 0.5 \times 0.41 = 0.205 \text{ m}$$

$$p_{geo} = E\frac{\Delta r}{{r_o}^2} = 150 \times \frac{0.205}{0.5^2} = 123 \text{ kPa}$$

Hence, the ultimate load the stone column can carry is:

$$Q_{u1} = k_p(p'_L + p_{geo})\pi {r_o}^2 = 4.2 \times (150 - 5 \times 9.81 + 123) \times \pi \times 0.5^2 = 739 \text{ kN}$$

As can be seen, the geotextile encasement more than doubles the ultimate load the stone column can carry.

Failure mechanism 2: Geotextile fails in hoop tension

The ultimate pressure the stone column can carry is calculated as:

$$p_{u2} = k_p\left(2G\frac{\Delta r}{r_o} + p_{geo\ f}\right)$$

where k_p is the passive earth pressure coefficient of the soil, G is the shear modulus of the soil outside the geotextile, $\Delta r/r_o$ is the relative increase in radius of the stone column, and $p_{geo\ f}$ is the confining pressure generated by the geotextile at failure. In this failure mechanism, the tensile strain of the geotextile at failure is calculated as:

$$\varepsilon = \frac{T}{E} = \frac{60 \text{ kN/m}}{150 \text{ kN/m}} = 0.4$$

Note that when the hoop strain of the geotextile is 0.4, the soil is approximately at the limit pressure (radial or hoop strain of 0.41), so in this fortuitous case, failure mechanisms 1 and 2 are the same.

Therefore, the ultimate load per stone column is 739 kN.

Problem 26.6

How would you make cement? How do you make lime? What is the difference between cement and lime?

Solution 26.6

Cement is made of calcium and silicon. To make cement in your kitchen, you mix powdered limestone (calcium carbonate, $CaCO_3$) and powdered clay (mostly silica SiO_2) and heat it to 1450°C; you will get a hard piece of rock. (Note that the oven in your kitchen is very unlikely to be able to generate this high a temperature.) If you then grind that piece of rock into a very fine powder, you will have a crude cement. When you add water to that very dry cement powder, an exothermic reaction (generates heat) called hydration takes places and produces calcium silicate hydrate, which is the main source of cement strength. Cement is the binder in concrete, mortar, and grout.

To make lime, take a piece of natural limestone rock ($CaCO_3$), heat it to about 1000°C to drive the carbon (CO_2) out of the limestone, and then grind the leftover piece of rock; you will have a white powder called lime or calcium oxide (CaO). If you then mix this white powder with a wet clay, it will hydrate, reabsorb carbon dioxide, and turn back into limestone.

The difference between cement and lime is that lime does not strengthen as rapidly as cement. It is weaker and more brittle than cement.

Problem 26.7

A soil has a D_{10} equal to 2 mm. The grout used to strengthen it has a D_{65} of 60 μm and a D_{95} of 130 μm. Can particulate grouting be successful?

Solution 26.7

$$N_1 = \frac{D_{10(soil)}}{D_{65(grout)}} \quad or \quad N_2 = \frac{D_{10(soil)}}{D_{95(grout)}}$$

According to one theory, grouting is feasible if $N_1 > 24$ and not feasible if $N_1 < 11$. According to another theory, grouting is feasible if $N_2 > 11$ and not feasible if $N_2 < 6$.

In this problem, grouting is feasible because:

$$N_1 = \frac{D_{10(soil)}}{D_{65(grout)}} = \frac{2\ \text{mm}}{60\ \mu\text{m}} = 33.3 > 24$$

$$N_2 = \frac{D_{10(soil)}}{D_{95(grout)}} = \frac{2\ \text{mm}}{130\ \mu\text{m}} = 15.4 > 11$$

Problem 26.8

A geosynthetic mat and column-supported embankment (GMCS) is used to build an embankment on soft clay. The embankment is 7 m high, built with a fill with a compacted unit weight of 20 kN/m^3 and a friction angle of 32°. The columns are 1 m in diameter with no pile cap and are placed on a square 2 m center-to-center grid. The bridging layer is 1 m thick with two layers of geosynthetic. Calculate the load per column, the tension in the geosynthetic layers due to spanning across the columns (T_1), and the tension due to lateral spreading (T_2). Assume that the net difference in stress on either side of the geosynthetic layer is 80% of the pressure under the embankment and that the geosynthetic has a modulus equal to 60 kN/m.

Solution 26.8

The load per column is:

$$Q_{col} = (\gamma H + q)A$$

$$Q_{col} = (20 \times 7 + 0) \times 2 \times 2 = 560\ \text{kN}$$

The tension T_1 due to the bridging effect between columns is given by:

$$6{T_1}^3 - (6T_1 - E_{GS})\left(\frac{\sigma_{net} \times A_{soil}}{p}\right) = 0$$

$$6{T_1}^3 - (6T_1 - 60)\left(\frac{0.8 \times 20 \times 7 \times \left(4 - \pi \times 0.5^2\right)}{\pi \times 1}\right) = 0$$

$${T_1}^3 - 114.6(T_1 - 10) = 0$$

which gives a tension T_1 equal to:

$$T_1 = -14\ \text{kN/m}$$

The tension T_2 due to the lateral spread of the embankment is given by:

$$T_2 = \frac{1}{2}K_a \gamma H^2 + qK_a H$$

$$K_a = \frac{1 - \sin\varphi}{1 + \sin\varphi}$$

$$K_a = \frac{1 - \sin 30}{1 + \sin 30} = \frac{0.5}{1.5} = 0.333$$

$$T_2 = \frac{1}{2} \times 0.333 \times 20 \times 7^2 + 0 = 163\ \text{kN/m}$$

So, the lateral spreading effect is much more severe than the bridging effect in this case.