CHAPTER
6

# IMPROVING SITE SOILS FOR FOUNDATION USE

## 6-1 INTRODUCTION

The centuries-old problem of land scarcity in the vicinity of existing urban areas often necessitates the use of sites with soils of marginal quality. In many cases these sites can be utilized for the proposed project by using some kind of soil improvement. This chapter will focus on several of the more widely used methods of improving soils for bearing capacity. An extremely large number of methods have been used and/or reported in the literature—many of which have been patented—and at an individual site one may use a mix of several methods to achieve the desired result. Chapter 12 will consider methods for increasing lateral stability.

For a given site a first step is to make a literature review of at least some of the methods reported. This together with a reasonable knowledge of geotechnical fundamentals allows the engineer to use either an existing method, a mix of methods, or some method coupled with modest ingenuity (unless limited by a governmental agency) to produce an adequate solution for almost any site.

Of principal interest in this chapter is the identification of means to obtain a significant increase in the bearing capacity of a soil. This can be achieved by altering the soil properties of $\phi$, cohesion $c$, or density $\rho$. Usually an increase in density (or unit weight $\gamma$) is accompanied by an increase in either $\phi$ or $c$ or both (assuming the soil is cohesive). Particle packing (compaction) always increases the density, with a resulting decrease in void ratio, and reduces long-term settlements. Particle packing usually increases the stress-strain modulus so that any "immediate" settlements are also reduced.

The rest of this section considers approaches to soil property modification.

**Mechanical stabilization.** In this method the grain size gradation of the site soil is altered. Where the site soil is predominantly gravel (say, from 75 mm down to 1 mm) binder material is added. *Binder* is defined as material passing either the No. 40 (0.425 mm) or No. 100

(0.150 mm) sieve. The binder is used to fill the voids and usually adds mass cohesion. Where the soil is predominantly cohesive (No. 40 and smaller sieve size) granular soil is imported and blended with the site soil.

In either case the amount of improvement is usually determined by trial, and experience shows that the best improvement results when the binder (or filler) occupies between 75 and 90 percent of the voids of the coarse material. It usually requires much more granular materials to stabilize cohesive deposits than binder for cohesionless deposits and as a result other stabilizing methods are usually used for clayey soils.

**Compaction.** This method is usually the most economical means to achieve particle packing for both cohesionless and cohesive soils and usually uses some kind of rolling equipment. *Dynamic compaction* is a special type of compaction consisting of dropping heavy weights on the soil.

**Preloading.** This step is taken primarily to reduce future settlement but may also be used to increase shear strength. It is usually used in combination with drainage.

**Drainage.** This method is undertaken to remove soil water and to speed up settlements under preloading. It may also increase shear strength since $s_u$, in particular, depends on water content. For example, consolidation without drainage may take several years to occur whereas with drainage facilities installed the consolidation may occur in 6 to 12 months.

**Densification using vibratory equipment.** Densification is particularly useful in sand, silty sand, and gravelly sand deposits with $D_r$ less than about 50 to 60 percent. This method uses some type of vibrating probe, which is inserted into the soil mass and withdrawn. Quality fill is added to the site to bring the soil surface to the required grade since the site soil usually settles around and in the vicinity of the vibrating probe.

**Use of in situ reinforcement.** This approach is used with stone, sand, cement, or lime columns. This treatment produces what is sometimes called *composite* ground. Sometimes small amounts of short lengths of plastic fibers or fiberglass can be mixed with the soil for strength improvement. The major precaution is to use a fiber material that has an adequate durability in the hostile soil environment.

**Grouting.** Initially this was the name for injection of a viscous fluid to reduce the void ratio (and $k$) or to cement rock cracks [see ASCE (1962)]. Currently this term is loosely used to describe a number of processes to improve certain soil properties by injection of a viscous fluid, sometimes mixed with a volume of soil. Most commonly, the viscous fluid is a mix of water and cement or water and lime, and/or with additives such as fine sand, bentonite clay, or fly ash.[1] Bitumen and certain chemicals are also sometimes used. Additives are used either to reduce costs or to enhance certain desired effects. Since the term *grout* is so loosely used in construction, the context of usage is important to define the process.

**Use of geotextiles.** These function primarily as reinforcement but sometimes in other beneficial modes.

**Chemical stabilization.** This means of stiffening soil is seldom employed because of cost. The use of chemical stabilizers is also termed *chemical grouting*. The more commonly used chemical agents are phosphoric acid, calcium chloride, and sodium silicate (or water

[1]A by-product from burning coal, primarily in electric power generating plants.

glass). Some laboratory tests indicate certain metallic powders (aluminum, iron) may produce beneficial effects as well [Hoshiya and Mandal (1984)]. ASCE (1957,1966) cited usage of an extremely large number of chemical grouting procedures (mostly patented, but most of the patents have probably expired by now).

Strictly, soil-cement and lime-soil treatment (often together with fly ash and/or sand) is a *chemical stabilization* treatment, but it is usually classified separately.

Several of the foregoing methods of soil improvement will be taken up in additional detail in the following sections. The primary emphasis, however, is on improving soils for use in building foundations. Additional background on the preceding methods may be obtained from the three ASCE conferences on "Soil Improvement," the latest being published by ASCE as Geotechnical Special Publication No. 12 (1987).

Appropriate references will be cited so the interested reader may obtain additional depth for a particular application.

## 6-2 LIGHTWEIGHT AND STRUCTURAL FILLS

A method that allows construction of relatively light structures (such as residences and one- or two-story structures with lightly loaded foundation slabs) on very soft base soil is to use either a lightweight fill or a carefully placed structural fill onto which the foundation is placed.

Lightweight fills may use expanded shale, certain industrial slags, and fly ash. A reduction in $\gamma$ from 18.5 to 16.5 kN/m$^3$ in a fill 1 m thick allows a 2 kPa foundation load increase for the same contact pressure of 18.5 kPa. These materials may be mixed with sand and/or gravel to produce a fill of the desired density and durability.

There are two "soft soil" cases to consider:

1. The site soil has such an extremely low shear strength that any surface load produces a shear failure (sinks into the mud). In this case it will be necessary to pretreat a surface thickness on the order of $150^+$ mm by sand (or a sand-gravel mixture) that is mixed with the top soil to produce a final mix with some load-supporting capacity.
2. The site soil has sufficient shear strength that it can support small surface loads.

For either of these cases a support fill is first placed by spreading imported fill to a loose depth between 0.5 and 1 m from hauling equipment as it is backed onto the site. Care is used that the soil underlying the fill is not much rutted in this operation. That is, the imported fill provides the necessary spreading of the hauler wheel loads to a pressure the underlying soil can support.

Lightweight or small spreaders are then used to bring the fill to the desired depth with minimal damage to the underlying soft base soil. Compaction is done with light- to medium-weight rollers once the layer (usually called *lift*) thickness is sufficient that the equipment weight does not cause the underlying soil to fail.

Construction of the building commences after the desired settlement has occurred under the *preload* of the fill. Vertical drains and a sand blanket beneath the fill may be used to speed consolidation. Fill thicknesses range from about 0.5 m to $1^+$ m.

Fills may be the most economical site improvement method available when used in conjunction with careful monitoring of the field work for floor slab-type buildings.

This method was used for a housing site near San Francisco on bay mud with an $s_u$ on the order of 15 to 25 kPa [Garbe and Tsai (1972)]. With a compacted fill of about 0.6 m the preload pressure on the underlying mud was on the order of 10 to 14 kPa. With the fill in place about 12 months prior to erecting the houses (using slabs on grade—no basements obviously), the soil consolidated sufficiently that the increase in $s_u$ could carry the building foundations and access roads.

Preloading, however, may not always produce a successful outcome. Duncan (1993—but see also "discussion" in 1995) described another housing development in the San Francisco Bay area where the outcome was rather uncertain. After about 12 years of preloading there was an estimate that subsequent differential settlement over a 23-m distance could approach 100 mm. The developer was required to provide an escrow account should later settlements require housing repairs.

Foundation loads from residential buildings are seldom over 15 to 20 kPa for wall footings and perhaps one-tenth of this for slabs on grade. Service roads should be of asphalt to allow deformation with minimal cracking and to allow repaving of bumps and potholes at minimum cost. It would also be necessary to stipulate a maximum truck load to avoid rutting.

## 6-3 COMPACTION

Compaction is usually an economical method of improving the bearing capacity of site soils. It may be accomplished by excavating to some depth, then carefully backfilling in controlled lift thicknesses, each of which is compacted with the appropriate compaction equipment. The backfill soil may be the excavated soil dried (or wetted) as necessary, possibly mixed with an admixture such as cement or lime, with or without fly ash or sand filler; or it may be imported soil from a nearby borrow pit. The standard compaction tests (ASTM, vol. 4.08) that may be used to establish the field density are these:

| **ASTM D 698** | **ASTM D 1557 (Modified)** |
|---|---|
| 24.4-N (5.5-lb) rammer | 44.5-N (10-lb) rammer |
| 305-mm (12-in.) drop | 457-mm (18-in.) drop |
| 944 cm$^3$ (1/30 ft$^3$) mold* | |
| 3 layers of soil | 5 layers of soil |
| 25 blows/layer | 15 blows/layer |

*Mold diameter = 101.6 mm for Methods *A* and *B* or 152.4 mm for Method *C*, which allows particles larger than 20 mm (3/4 in.).

The foregoing procedures are for ASTM test Methods *A* and *B*, which are for soil with grains smaller than 10 mm ($\frac{3}{8}$ in. nominal). Refer to the ASTM test Method *C* if larger soil particles are used.

The modified compaction test (D 1557) just listed is not used much in building construction since there is seldom enough soil improvement to justify the additional compaction effort and necessary quality control. Figure 6-1 presents typical compaction curves for several soils obtained using Method *A* from both ASTM standards.

For fills that will later support any structure it is usual to perform compaction tests to establish the required compacted density and *optimum moisture content* (OMC) for the field

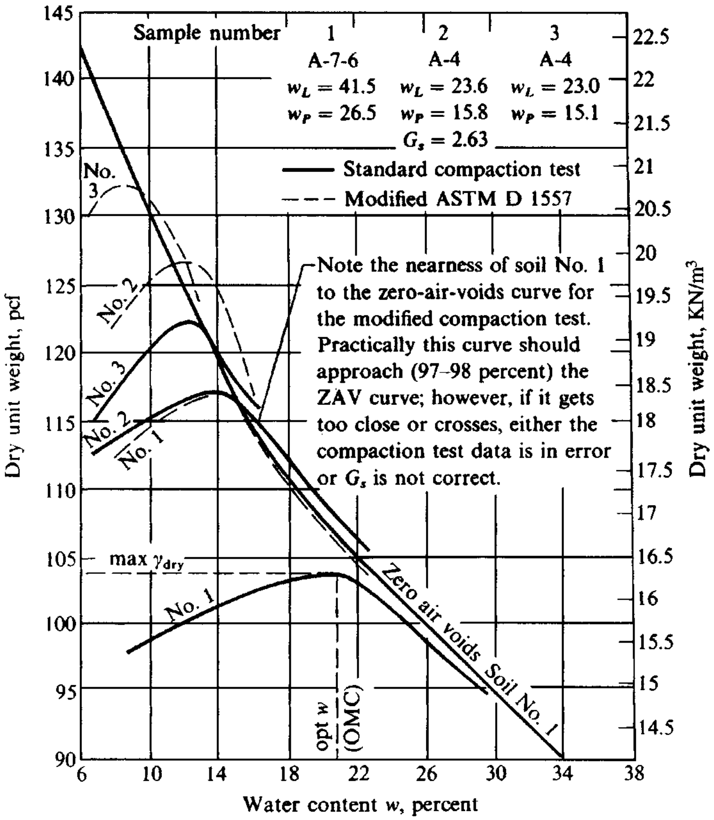



**Figure 6-1** Typical compaction curves for three soils classified as indicated on the graph and by both standard (ASTM D 698) and modified (ASTM D 1557) methods. The zero-air-voids (ZAV) curve is shown only for soil sample no. 1.

soil. Field density tests (quality control) are then performed to ensure the desired unit weight $\gamma$ is obtained. With compaction control, the fill is often of better quality than the underlying soil. The underlying soil will undergo settlements of varying magnitude depending on its characteristics and the depth of fill $D_{\text{fill}}$ which produces a settlement/consolidation pressure of $\gamma D_{\text{fill}}$.

Settlements will be *nonuniform* if the fill depth varies or if the site consists of both cut and fill. Settlements may be of long duration unless special steps are taken to speed up the process such as overfill (or *preloading*) to increase the settlement pressure and/or installation of drainage to speed consolidation.

Compaction of cohesive soils can be accomplished using sheeps-foot or rubber-tired rollers. Lifts are commonly 150 to 200 mm thick. It may be necessary either to aerate the soil by disking to reduce the water content or to add water from mobile water tanks if the field $w_N$ is too low. Minimum compaction effort is required when the field $w_N$ is near (or at) the OMC.

Compaction of cohesionless soils can be accomplished using smooth wheel rollers, commonly with a vibratory device inside, so the compaction is a combination of confinement, pressure, and vibration. Lift depths up to about 1.5 to 2 m can be compacted with this equipment. Better results are obtained, however, for lift thicknesses of 0.6 to 1 m. Where there is an ample supply of water and its use does not adversely affect the surrounding soil, flooding (100 percent saturation) will substantially reduce the required compaction effort—particularly if the in situ sand is slightly damp where surface tension impedes densification.

In confined spaces, it is necessary to use hand-powered equipment for compacting the soil. This requirement reduces the lift thickness so that if density has been specified, lift thicknesses should not exceed 75 to 100 mm. For lifts that are too thick, compacting by hand—or any method—results in a dense upper crust overlying uncompacted soil which will later settle under self-weight and/or applied load, regardless of the type of equipment used or soil location.

Specific details of compaction methods and equipment necessary to compact various soils, laboratory tests to establish compaction specifications, and field tests for verification are beyond the scope of the overview presented here. The interested reader may wish to consult publications (with included references) such as these:

"Criteria for Compacted Fills," Building Research Advisory Board, National Academy of Sciences, Washington, DC, 1965.

*Symposium on Compaction of Earthwork and Granular Bases,* Highway Research Record no. 177, National Academy of Sciences, Washington, DC, 1967.

*Soil Compaction and Corrugations,* Highway Research Record no. 438, National Academy of Sciences, Washington, DC, 1973.

"Compacted Clay: A Symposium," *Trans. ASCE,* vol. 125, pp. 681-756, 1960.

"Sand Compaction with Vibratory Rollers," D'Appolonia, D.J., et al., *JSMFD,* ASCE, vol. 95, no. 1, January, pp. 263–284, 1969.

One of the recent textbooks on Geotechnical Engineering such as Bowles (1984).

Although these references are somewhat dated, this soil improvement method was one of the earliest that was heavily researched. There is little that can be added to the current knowledge base.

The bottom of any footing trench or basement excavation should always be compacted either using hand or full-size equipment. Although this precaution does not recover heave (expansion due to loss of overburden) it does place the base soil loosened by the excavation equipment into a dense state.

### 6-3.1 Consolidation Settlements of Compacted Fill

Early geotechnical engineers knew that a compacted fill would undergo some subsidence due to self-weight producing grain readjustment and/or some squeezing (or creep). Any consolidation settlements were supposed to be developed in the underlying soil supporting the fill. It is now known that both the underlying soil *and* the compacted fill undergo consolidation. The consolidation of the underlying soil is similar to that described in Secs. 2-10 and 5-12.

Fill settlements can range from about 60 mm to well over 500 mm depending on the depth and the following factors:

1. Soil fabric (how much particle packing, type of particles, etc.); related to the compaction effort.

2. The compaction water content, and later change in water content. Factors 1 and 2 are, of course, related.
3. The fill height (or depth for a self-weight component) and any surcharge or applied vertical stress as from a foundation.

The fill consolidation usually involves later mass saturation and may occur a number of years after construction. After a long period of time the vertical movements may not always be correctly attributed to consolidation settlement within the fill.

It appears that one might estimate the probable consolidation settlement in the fill by compacting soil samples in the laboratory to the field density and field compaction water content. These samples can then be put into a consolidation test device, saturated, and then tested for swell and for both primary and secondary compression. It may be necessary to use back-pressure to speed the saturation process if the consolidation device allows it. Some of this methodology is described by Brandon et al. (1990) and Lawton et al. (1989).

## 6-3.2 Dynamic Compaction

A widely used method of compaction using a mobile crane to lift and drop a heavy tamper onto the soil is called *dynamic compaction* (some persons call the procedure *dynamic consolidation*). Although the dropping of a weight on the soil had probably been in use for centuries, it was reintroduced to the profession and patented by L. Ménard in France ca. 1970 [see Ménard and Broise (1975)]. Compaction can be achieved to a substantial depth depending on weight (or mass) of the tamper, height of fall, and the type of soil.

Although the dynamic compaction tamper can have a mass up to 150,000 kg (or 150 tonnes), the usual mass is on the order of 10 to 20 tonnes and is dropped from heights ranging up to 40 m (usually 10 to 20 m) onto a grid spacing so that the site requiring improvement is adequately covered. Craters ranging from 0.5 to 2 m in depth are produced at the points of impact.

After a selected part of the area to be compacted is covered by a pass (drop in each grid point) it is graded with a bulldozer using imported fill as necessary to smooth the surface, the next pass is made, and so on until the desired density is obtained. Density is usually specified based on before and after penetration tests (either SPT or CPT). After the site improvement is completed, the area is brought to grade and compacted with ordinary compaction equipment.

Most saturated soils that can be classified through silty and/or clay sands and gravels can be considerably improved by this method. The amount of compaction tends to decrease with an increase in silt or clay content. Saturated clays tend toward almost no improvement because the impact results in an instantaneously high pore pressure, an immediate loss of shear strength, and remolding. Partially saturated clays may be improved, at least in the region above the GWT.

In practice several trial grid sections are used to determine the optimum drop spacing, drop weight (and/or height of fall), and number of drops.

For cohesionless soils Leonards et al. (1980) suggested the depth of compaction influence $D_i$ is approximately

$$D_i = \frac{1}{2}\sqrt{Wh} \qquad \text{(m)} \tag{6-1}$$

In cohesive soils Ménard and Broise (1975) suggested

$$D_i = \sqrt{Wh} \qquad \text{(m)} \tag{6-2}$$

where $W$ = mass of tamper in tonnes (1 tonne = 1000 kg)
$h$ = height of fall, m

Both of these equations are in current use.

Mayne et al. (1984) give a review of a large number of sites where dynamic compaction was used; Rollins and Rogers (1991) present a more recent example of the method for a collapsible alluvial soil. See Greenwood and Thomson (1984) for additional dynamic compaction details if necessary.

It is evident that the improvement will range in quality from the point of impact and grade into untreated soil at the depth $D_i$. The depth $D_i$ should be on the order of $2B$ of the least lateral foundation dimension for smaller bases, but engineering judgment and available equipment will determine the influence depth $D_i$ for large bases such as mats that cover large foundation areas. Grid spacings are commonly on the order of 1.5 to 4 meters.

Ordinarily, dynamic compaction/consolidation is only economical when

1. Site plan involves an area of some 5000 to 10 000 m$^2$.
2. Depth of soil is too great to use excavation and replacement.
3. Impact vibrations that are on the order of 2 to 12 Hz will not cause damage to nearby developments.

Where the water table is near the ground surface or there is a soft clay surface deposit, it may be necessary first to lay a free-draining granular blanket on the order of 200 to 1000 mm thick over the area to be dynamically compacted.

## 6-4 SOIL-CEMENT, LIME, AND FLY ASH

In many cases where slab-on-grade construction is to be used the most economical solution to increase the bearing capacity may be to do one of two things.

1. Use soil-cement, with or without a sand or fly ash filler. In this procedure soil samples are mixed with varying percentages of cement and/or sand and/or fly ash, cured in a manner somewhat similar to concrete control test cylinders,[2] and tested to obtain the unconfined compression strength $q_u$. That mix providing the required strength becomes the job mix. The cement and/or other admixtures are either deposited on the soil and thoroughly mixed at the necessary water content with discs and similar farm equipment or run through a traveling soil processor where the chemicals and water are added, blended and redeposited on the soil for grading and compaction. Depths to about 1.5 m can be treated in this manner;

[2] ASTM has a number of standards relating to "soil-cement".

greater depths usually require some alternative method. The required cement by weight is seldom over 5 percent.

2. Use lime or a mix of lime and sand, with or without fly ash, in a manner similar to soil-cement.

## 6-5 PRECOMPRESSION TO IMPROVE SITE SOILS

A relatively inexpensive, effective method to improve poor foundation soils in advance of construction of permanent facilities is *preloading*. The preload may consist of soil, sand, or gravel; and in the case of oil or water tanks, gradual filling of the tanks may be used for the preload. Sometimes the preload may be accomplished by lowering the groundwater table. It may also be accomplished by "ponding," that is, building a watertight containment that is filled with water [but requires protection against vandalism and unauthorized recreation (such as swimming)].

How or what to use to accomplish preloading will be determined by relative economics. Aldrich (1965) [see also Johnson (1970)] conducted a survey among several organizations to produce a report on preload practices that were current at that time.

Precompression (or preloading) accomplishes two major goals:

1. Temporary surcharge loads are used to eliminate settlements that would otherwise occur after the structure is completed.
2. Preloading improves the shear strength of the subsoil by increasing the density, reducing the void ratio, and decreasing the natural water content $w_N$.

Preloading is most effective on normal to lightly overconsolidated silts, clays, and organic deposits. If the deposits are thick and do not have alternating sand seams, the preloading may necessitate using sand drains (see Sec. 6-6) to reduce the time necessary to effect consolidation.

The amount of settlement eliminated by using preloading should be 100 percent of primary consolidation. As much secondary compression is removed as practical so that, in combination with the eliminated settlement, any remaining after project completion will be tolerable. The primary consolidation can be computed by obtaining the stress increase using the Boussinesq method of Chap. 5 for several points beneath the loaded area and using Eq. (2-44). The secondary compression may be estimated using Eq. (2-49) repeated here, expanded, and terms reidentified to obtain

$$\Delta H_s = \frac{C_\alpha H}{1 + e_o} \log \frac{t_f - t_i}{t_i} \tag{6-3}$$

where $\Delta H_s$ = secondary compression settlement, in units of $H$
$H$ = thickness of stratum in field, m
$C_\alpha$ = coefficient of secondary compression
$t_f$ = time of interest when $\Delta H_s$ occurs, days or years
$t_i$ = time at the end of primary consolidation or slightly later, days or years.

The total settlement for the preload is the sum of the primary and secondary settlements [the sum of Eqs. (2-44) and (6-3)].

Shear strength tests before and after preloading are necessary to evaluate the improvement in strength with preconsolidation. These are best run on undisturbed tube samples in either unconfined or triaxial tests. The in situ vane may not give much indication of any shear strength improvement, for the vane measures horizontal rather than vertical shear strengths. Since the lateral improvement is likely to be on the order of $K\sigma_v$, with $K$ usually less than 0.5, preload improvement that may be sufficient for vertical loads may be too small to be detected by the shear vane test with sufficient accuracy or reliability to be of value [Law (1979)].

Normally the preload surcharge would be greater than the estimated weight of the proposed structure so that postconstruction settlements are negligible. There may be some rebound and recompression as any preload is removed and before the building load is applied.

Preloading does not seem to be much used at present since a number of other procedures can be used to improve the soil that are comparable in cost, allow more rapid access to the site, and do not require disposal of the excess preload material.

In extremely soft cohesive and peaty deposits such as glacial lakes, river deltas, and peat bogs a procedure called *displacement preloading* may be used where haulers back to the site edge and dump the quality fill. The fill load induces a shear failure in the in situ soil, which causes it to displace laterally away from the fill. The lateral displacement usually produces viscous waves in the soil called mud waves. When there is enough fill accumulated it is compacted and the process continued until the desired area is stabilized. This procedure is of use for shoreline construction and has been used to produce causeways across lakes for railroad tracks and roadways.

## 6-6 DRAINAGE USING SAND BLANKETS AND DRAINS

When either a fill or a soil preload is placed on a saturated cohesive deposit, the length of the drainage path may be increased—perhaps to the top of the fill. Since the length of the drainage path determines the time for consolidation, this should be as short as possible.

When the water table is very near the ground surface, either the site should be graded so it slopes to one side or a series of shallow collection ditches should be cut. Next a layer of sand (called a *sand blanket*) 100 to 150 mm thick is placed on top of the site and in the drainage ditches, and then the preload. Water squeezed from the soil being consolidated then flows up to the ditches or sand blanket and drains to the edge for disposal. This will greatly speed the drainage process, since the coefficient of permeability is larger in sand.

### 6-6.1 Sand Drains

We can extend this concept further and install vertical columns of sand at selected intervals in the existing soil. Under the hydraulic gradient produced by the fill (or preload) the water flows from a higher to a lower energy potential. Since the water can move faster through the sand than through the in situ soil, the sand columns (sand drains) become points of low energy potential.

Maximum flow rate is obtained by incorporating a sand blanket with the sand drains. Sand drains can be installed even where the consolidating stratum is some depth below the surface to speed up the consolidating process. Here, however, it may not be desirable or necessary to use a sand blanket.

Consolidation theory of Sec. 2-10 is the basis for both sand blankets and sand drains. The time $t_c$ for consolidation is estimated from a rearrangement of Eq. (2-38) to obtain

$$t_c = \frac{TH^2}{c_v} \tag{6-4}$$

The dimensionless factor $T$ depends on the percent consolidation $U$ (see Table 2-4) and is about 0.848 and 0.197 for 90 and 50 percent consolidation, respectively. The coefficient of consolidation $c_v$ is usually back-computed from a consolidation test by solving Eq. (6-4) for $c_v$. The coefficient is also

$$c_v = \frac{k}{\gamma_w m_v} \qquad \text{(2-35)}$$

where all terms have been defined in Chap. 2. For radial drainage as in sand drains, the coefficient of permeability (or *hydraulic conductivity)* $k$ in Eq. (2-35) would be the horizontal value, which is often four or five times as large as the vertical value.

The theory of radial drainage into sand drains, including allowance for "smear" effects on the sides of the holes from soil on the auger flights that reduce inflow, has been presented by Richart (1959) [see also Landau (1978)]. Since one is fortunate to determine the order of magnitude of $k$ (the exponent of 10), for practical purposes the time for consolidation of a layer can be computed as follows:

1. Take $H = \frac{1}{2}$ the longest distance between sand drains, m.
2. Compute $c_v$ using Eq. (2-35) with $k$ = horizontal coefficient of permeability (or your best estimate of that value), m/day.
3. Use $T$ from Table 2-4 for the appropriate percentage of consolidation. For 90 percent consolidation use $T = 0.848$.
4. Solve Eq. (6-4) for $t_c$ in the time unit of days.

The calculated time will be somewhat in error from factors such as vertical drainage within the consolidating layer, presence of thin sand seams, one- or two-way vertical drainage, how the distance $H$ compares with the clay thickness, etc.

Sand drains are installed by several procedures in diameters ranging from 150 to 750 mm. Landau (1966) describes several that are still current:

1. *Mandrel-driven pipes.* The pipe is driven with the mandrel closed. Sand is put in the pipe, which then falls out the bottom as the pile is withdrawn, forming the drain. Air pressure is often used to ensure continuity and densify the sand.
2. *Driven pipes.* The soil inside is then removed using high-pressure water jets. The rest of the procedure is the same as method 1.
3. *Rotary drill.* A casing is used as required, then the boring is filled with sand. Any casing used is pulled as the boring is filled. The sand may be rammed as necessary to increase its density, producing some enlargement of the column over the drilled diameter.
4. *Continuous-flight hollow auger.* The sand may be introduced using air pressure through the hollow stem to fill the cavity as the auger is withdrawn.

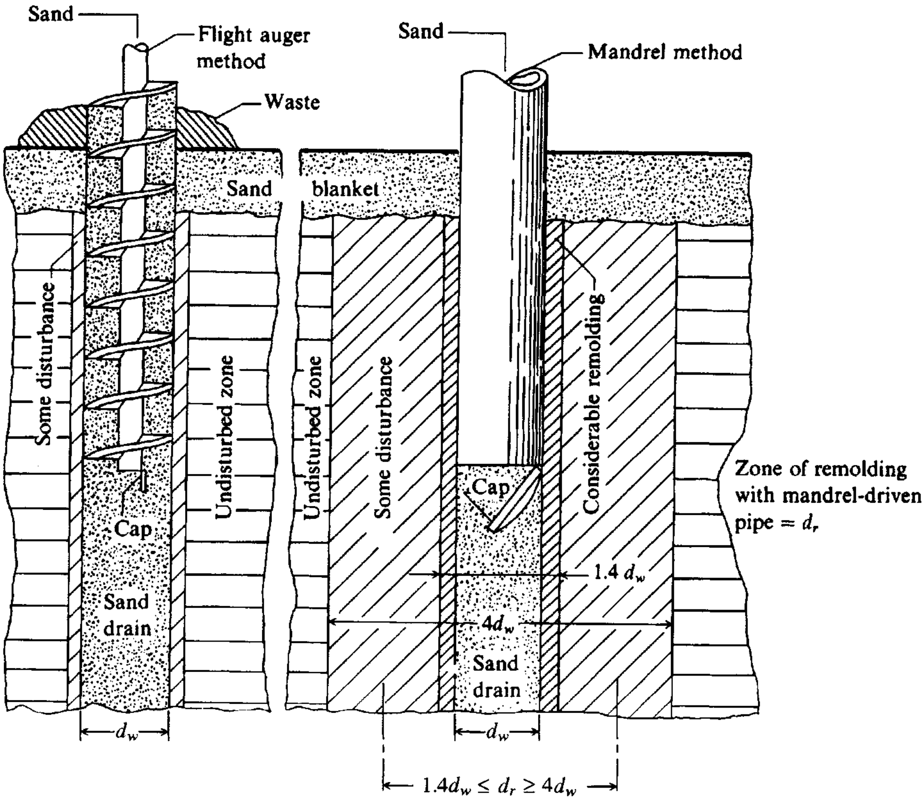



**Figure 6-2** Two commonly used methods of constructing sand drains. [Landau (1966).]

Figure 6-2 illustrates methods 1 and 4.

Note that if we construct a pattern of sand drains using displacement-driven columns and then later construct the interior drains (also using displacement columns) the site drainage should be much more rapid since the excess pore pressure produced when installing the interior drains can drain laterally into the existing drains as well as back into the just-installed drains.

Soil drainage is related to settlement (volume change), and the larger the settlement under preload, the less to be expected when the structure is built. Drainage is also related to the change in the natural water content since a change in void space results in a permanent change in water content for saturated soils. The change in water content is also a measure of the improvement in the undrained shear strength $s_u$.

## 6-6.2 Wick Drains

Wick drains are now being widely used in lieu of sand columns for soil drainage. A wick drain is a geotextile consisting of a grooved plastic or paper core covered by plastic or paper membranes to produce a "wick" ranging from about 100 to 300 mm wide × 4 to 6 mm thick and of the necessary length. The membrane cover provides a permeable soil barrier to reduce core clogging. The core provides a ready conduit to the surface into a sand or textile filter blanket or into horizontal trench drains.

The particular attraction of wick drains is economy since installation costs per meter are typically one-quarter to one-fifth those of sand drains. They can be installed to depths up to 30 m using a conventional vibratory hammer (as used for pile driving) and a special wick installation rig. According to Morrison (1982) wick drains have about 80 percent of the soil consolidation market—probably about 80–85 percent in 1995. Several references and some design theory on wick drains are cited by Koerner (1990). For current materials consult recent issues of the "Geotechnical Fabrics Report" published monthly by the Industrial Fabrics Association International (see footnote 5 on p. 368).

The same approximate equations for sand drains can be used for wick drains to establish spacing and estimate time for consolidation to occur.

Wick drains provide no strengthening effect on the soil (unless they are laid horizontally) except for that resulting from the reduced water content and for the void ratio reduction that may result from any increase in effective stresses within the soil mass.

Note that the drainage process can be considerably speeded by installing mandrel-driven pipe displacement sand drains interior to the peripheral wicks.

## 6-7 SAND COLUMNS TO INCREASE SOIL STIFFNESS

Outside the United States—particularly in the Asian and Pacific Rim regions—sand columns are widely used to increase soil stiffness in both sand and clay deposits. Soil stiffness (or improvement) is directly related to the increase in either the SPT blow count $N$ or the CPT cone resistance $q_c$. That is, if the initial soil resistance ($N$ or $q_c$) is too low to give an adequate bearing capacity, sand columns might be an economical solution, i.e., use the $N$ after installing the columns for computing the bearing capacity.

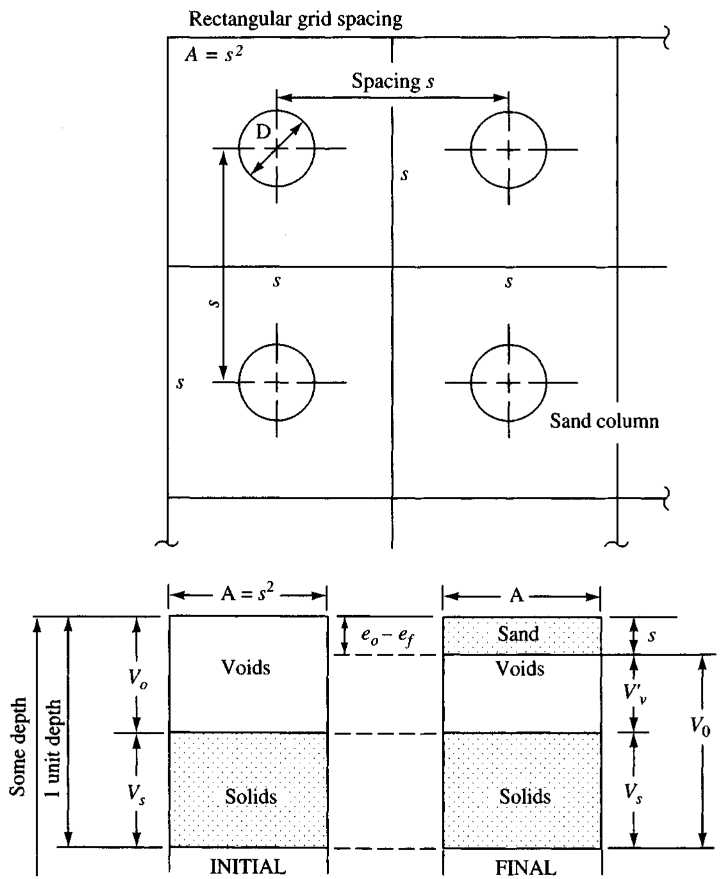
The use of sand columns is mostly a trial-experience combination process where their use is appropriate. That is, a trial spacing is chosen and sand columns are inserted. Sand columns are usually drilled at diameters $D_o$ between 600 and 800 mm, but after construction the actual column diameters $D_f$ range from 1.5 to $1.6D_o$. Column depths usually range from about 3 to 8 m but depend on site and purpose.

The before and after stiffness is measured along with the amount of sand needed to produce the required end product. That spacing and/or column density producing the required degree of soil improvement is then specified for that site. Barksdale and Takefumi (1991) cite some equations (see Fig. 6-3) that attempt to quantify some of this process, but the several assumptions used make it necessary to always verify the improvement using either the SPT or CPT. It is also necessary for contractor payment to measure the actual volume of sand used.

To quantify a project approximately one would make a best estimate of the current in situ void ratio $e_o$. Next one would make an estimate of the final void ratio $e_f$ based on available information or by simply deciding the void ratio should be some value.

You should refer to the previously cited reference for the use of sand columns to strengthen clay deposits.

Stone columns can also be used in sand deposits and they are constructed in a similar manner. Their use is not recommended in sand, however, since the sand column can be constructed more economically. The reason is that the in situ sand can be used as the primary source for the column material, which can then be supplemented with a smaller amount of imported material, whereas the full volume of the stone column would have to be imported.




For the initial conditions:

$$e_o = \frac{V_v}{V_s} \rightarrow V_v = e_o V_s$$

The initial volume is

$$\begin{aligned} V_o &= V_s + V_o \\ &= V_s + e_o V_s \\ &= V_s(1 + e_o) \end{aligned}$$

For the final conditions:

Noting that the total volume is now $V_o = V_s + V'_v + s$ we can find the sand volume $\bar{s}$ per unit of depth by proportion as follows:

$$\frac{\bar{s}}{e_o - e_f} = \frac{V_v}{e_o} \rightarrow \bar{s} = \frac{V_v}{e_o}(e_o - e_f)$$

The sand ratio as per unit treatment (sand column) depth is:

$$a_s = \frac{\bar{s}}{A}$$

A tentative column spacing distance $s$ for square grid (as shown in the figure) is $s = \sqrt{V_o}$

**Figure 6-3** Sand columns for soil strength improvement. [After Barksdale and Takefumi (1991).]

**Example 6-1.** We have somehow found $e_o = 0.8$ in a sand deposit and have estimated the desired $e_f = 0.5$ and a trial grid spacing of 3 m.

***Required.*** Make an estimate of the amount of sand fill required per meter of improvement depth $D_i$.

***Solution.*** For this problem we have $A = 3 \times 3 = 9\ \text{m}^2$ and for a 1-m depth,

$$V_o = 9 \times 1 = 9\ \text{m}^3$$

From $V_o = V_s + e_o V_s = V_s(1 + e_o)$ (see Fig. 6-3) we obtain

$$V_s = 9/1.8 = \mathbf{5}\ \text{m}^3$$

The original $V_v = V_o - V_s = 9 - 5 = \mathbf{4}\ \text{m}^3$. The theoretical volume of sand required *per meter of depth* is

$$\bar{s} = \frac{V_v}{A}(e_o - e_f) = \frac{4}{0.8}(0.8 - 0.5) = 5(0.3) = \mathbf{1.5}\ \text{m}^3$$

Still to be determined is the drill diameter, the depth of the sand column, and whether a final void ratio $e_f = 0.5$ is obtainable.

////

## 6-8 STONE COLUMNS

If, instead of using sand for the column, gravel or stones are used, the result is a *stone column*. The vibratory devices or procedure no. 1 used to install sand drains and sand columns can also be used to insert gravel or stone columns into the soil. The granular material commonly ranges in gradation from about 6 to 40 mm ($\frac{1}{4}$ to $1\frac{1}{2}$ inches).

Stone columns may be used in sand deposits but have particular application in soft, inorganic, cohesive soils. They are generally inserted on a volume displacement basis, that is, a 600- to 800-mm diameter hole is excavated to the desired depth $L_c$. The depth may be on the order of 5 to 8 m, and sometimes the hole requires casing to maintain the shaft diameter. Stone is introduced into the cavity in small quantities and rammed (while simultaneously withdrawing any casing). The rammed stone increases the drilled diameter of the stone column shaft, and it is necessary to record the hole depth $L_c$ and volume of stone $V_c$ used for the column so that the final nominal shaft diameter can be approximately computed. The lateral expansion of the column due to ramming will induce excess pore pressures in clay, but these rapidly dissipate back into the much larger voids in the granular column. The net effect is to produce a fairly rigid vertical stone mass (the stone column) surrounded by a perimeter zone of somewhat stronger material which has a slightly reduced void ratio. This insertion method also ensures intimate contact between soil and column.

The vibroflotation (see Fig. 6-6) method can be used to produce a stone column by sinking the device, backfilling the cavity with stone, and then raising and lowering the vibroflot while adding additional stone. The result is a densely compacted stone column of some depth with a diameter on the order of 0.5 m to 0.75 m.

Similarly, a closed end pipe mandrel can be driven to the desired depth and a trip valve opened to discharge the stone. Either a rammer packs the soil through the pipe as it is withdrawn and with stone added as needed, or the mandrel is withdrawn until the valve can be closed and this used to ram against the stone to expand and densify the column.

Stone columns are spaced from 1.2 m to about 3 m on center on a grid covering the site. There is no theoretical procedure for predicting the combined improvement obtained, so it is usual to assume that the foundation loads are carried only by the several stone columns with no contribution from the intermediate ground. Work on pile caps by the author indicates that this is reasonable when the stone columns are more than about 10 times as stiff as the surrounding soil. Also a compacted layer of granular material should be placed over the site prior to placing the footings.

An approximate formula for the allowable bearing pressure $q_a$ of stone columns is given by Hughes et al. (1975)

$$q_a = \frac{K_p}{\mathrm{SF}}(4c + \sigma_r') \tag{6-5}$$

where $K_p = \tan^2(45° + \phi/2)$

$\phi'$ = drained angle of internal friction of stone

$c$ = either drained cohesion (suggested for small column spacings) or the undrained shear strength $s_u$ when the column spacing is over about 2 m

$\sigma_r'$ = effective radial stress as measured by a pressuremeter (but may use $2c$ if pressuremeter data are not available)

SF = safety factor—use about 1.5 to 2 since Eq. (6-5) is fairly conservative

The allowable load $P_a$ on the stone column of average cross-sectional area $A_c = 0.7854D_{\mathrm{col}}^2$ is

$$P_a = q_a A_c \tag{6-5a}$$

where $q_a$ = allowable bearing pressure from Eq. (6-5)

We can also write the general case of the allowable column load $P_a$ as

$$P_a = (c_s A_s + A_c c_p N_c) \cdot \frac{1}{\mathrm{SF}} \tag{6-5b}$$

where $c_s$ = side cohesion in clay—generally use a "drained" value if available; $c_s$ is the side resistance ($\gamma z K \tan\delta$) in sand

$c_p$ = soil cohesion at base or point of stone column

$A_s$ = average stone column perimeter area

To compute $A_s$, use the in-place volume of stone $V_c$ and initial column depth $L_c$ as follows:

$$A_c L_c = 0.7854 D_{\mathrm{col}}^2 L_c = V_c \qquad \text{and} \qquad D_{\mathrm{col}} = \sqrt{\frac{V_c}{0.7854 L_c}}$$

$$A_s = \pi D_{\mathrm{col}} L_c$$

Observe that, by using the volume of stone $V_c$, the diameter $D_{\mathrm{col}}$ computed here is the *nominal* value. In Eq. (6-5*a*),

$N_c$ = bearing capacity factor as used in Chap. 4, but use 9 for clay soils if $L_c/D_{\mathrm{col}} \geq 3$ (value between 5.14 and 9 for smaller $L/D$)

The allowable *total* foundation load is the sum of the several stone column contributions beneath the foundation area (perhaps 1, 2, 4, 5, etc.).

Stone columns should extend through soft clay to firm strata to control settlements. If the end-bearing term ($A_c c_p N_c$) of Eq. (6-5*b*) is included when the column base is on firm strata, a lateral bulging failure along the shaft may result. The bulge failure can develop from using a column load that is too large unless the confinement pressure from the soil surrounding the column is adequate. The failure is avoided by load testing a stone column to failure to obtain a $P_{\text{ult}}$ from which the design load is obtained as $P_{\text{ult}}/\text{SF}$ or by using a large SF in Eq. (6-5*b*) or by not including the end-bearing term (now one can use a smaller SF).

Taking this factor into consideration gives a limiting column length $L_c$ (in clay based on ultimate resistance) of

$$P_{\text{ult}} \leq \pi D_{\text{col}} L_c c_s + 9 c_p A_c \qquad A_c = 0.7854 D_{\text{col}}^2$$

Solving for $L_c$, we obtain

$$L_c \geq \frac{P_{\text{ult}} - 7.07 d_p D_{\text{col}}^2}{\pi D_{\text{col}} c_s} \tag{6-6}$$

where all terms have been previously identified.

Settlement is generally the principal concern with stone columns since their bearing capacity is usually quite adequate. No method is currently available to compute settlement on a theoretical basis. Settlements are estimated on the basis of empirical methods, of which Fig. 6-4 is typical. From this figure we see that stone columns can reduce the settlement to nearly zero depending on column area, spacing, and initial soil strength.

Note that any substantial improvement in settlement may require placing a granular surcharge over the treated area and rolling it prior to placing the foundation. A surcharge may be necessary because the upper column depth to approximately 0.6 m is often somewhat loose from the placing process and if not compacted may allow an unacceptable settlement.

Stone columns are not applicable to thick deposits of peat or highly organic silts or clays.

## 6-9 SOIL-CEMENT PILES/COLUMNS

The soil-cement pile (or column), SCP, is a relatively recent innovation for soil improvement that uses a special (proprietary) soil drill bit. The drill bit advances into the soil, cutting and grinding the soil and simultaneously injecting the cement (and any additives) slurry into the cuttings. A shear (or fixed) blade somewhat larger than the hole diameter is located above the drill head and is fixed into the sides of the boring to keep the soil between the drill and shear blade held in place so that it can be well-mixed with the cement slurry (see Fig. 6-5*a*). When the column depth is reached a soil-cement pile has been formed; the drill is withdrawn, with the counterrotation further blending the soil cuttings with the injected cement slurry.

The process is extremely rapid and SCP diameters from 0.6 to 1 m can be readily produced in lengths varying from about 1.5 to 10 m, but maximum depths to 35 m are possible. A typical side view of an SCP is shown in Fig. 6-5*b*.

The design process is as follows:

1. Obtain representative samples of the soil to be improved, including unconfined compression $q_u$ and/or SPT blow counts $N$.

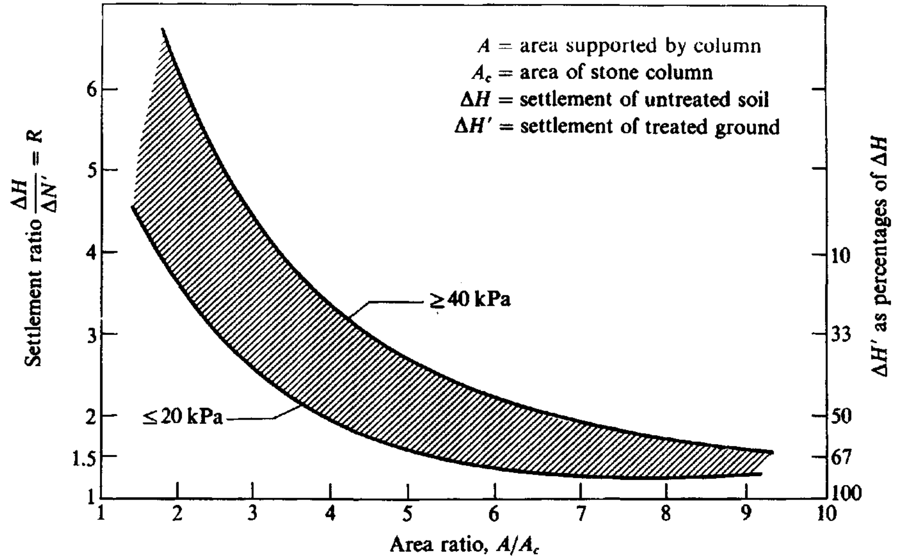



**Figure 6-4** Approximate settlement reduction for ground reinforced with stone columns. [After Greenwood and Thomson (1984).]

**Example**

Stone columns in soil with $s_u = 25$ kPa
Average column diam. = 1 m
Average column spacing = 2 m center-to-center
$\Delta H$ of untreated ground estimated at 125 mm

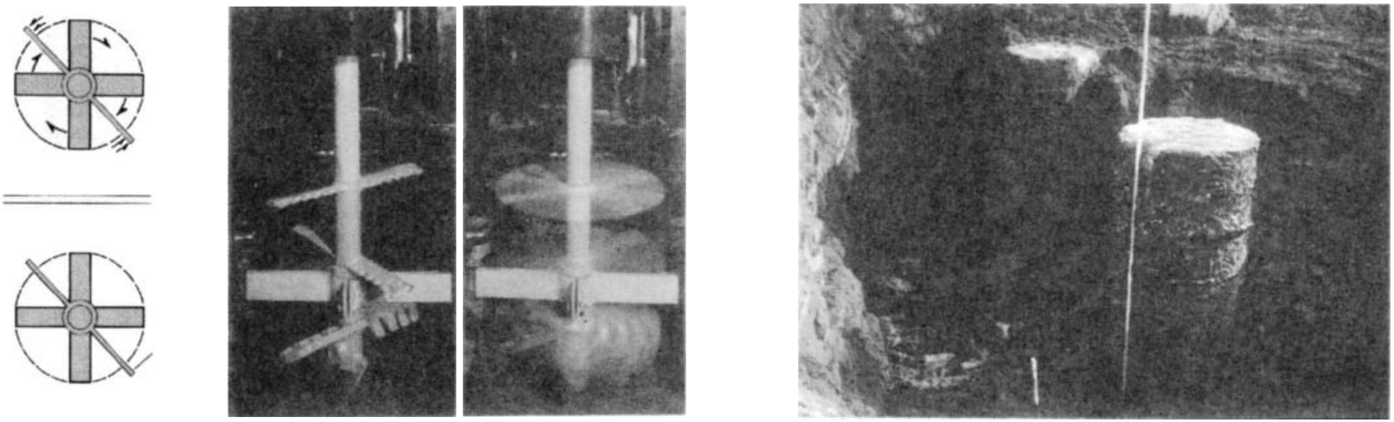
*Required:* Estimate $\Delta H'$ of treated ground

$$A_c = 0.7854(1)^2 = 0.7854$$
$$A = 2 \times 2 = 4$$
$$A/A_c = 4/0.7854 = 5.09 \text{ use } 5.1$$

From figure interpolating into hatched zone at $s_u = 25$ obtain

$$R = \Delta H/\Delta H' = 2 \text{ (or 50\%)}$$
$$\Delta H' = 125/2 = 125(0.5) = 65 \text{ mm}$$

Note generous rounding since method is inexact.



(*a*) Proprietary soil-cement pile drill

(*b*) Side view of a 1.52 m partially excavated SCP

**Figure 6-5** Soil-cement piles. (Photos courtesy O. Taki, SCC Technology, Inc., Belmont, CA.)

2. Mix soil samples with different amounts of cement slurry and produce soil-cement cylinders, which are cured as for any type of soil-cement project. Refer to ASTM D 1633 for compressive strength tests and to D 2901 for cement content.
3. From cylinder compression tests determine the appropriate cement-slurry proportions (water-cement ratio) and slurry injection per unit volume of pile.
4. When the SCPs have been installed and cured, obtain sufficient cores to ascertain the unconfined compression core strength to verify quality.

Soil-cement piles may be used alone or, more commonly, in a closely spaced line to form a wall to maintain an open excavation or basement space. If the spacing produces pile overlap or the spacing is such that a jet-grout operation (of Sec. 6-10) can fill the space between any two piles, a nearly water-tight wall can be formed. Basically a SCP wall [see Taki (1992)] consists in obtaining the unconfined compression strength of the soil-cement cylinders $q_{sc}$. The unconfined shear strength is taken as

$$s_{u,sc} = \tfrac{1}{2} q_{sc} \qquad \text{(same as for soil)}$$

The allowable compressive strength for column design (without any reinforcement) is taken as

$$f_{c,sc} = \frac{s_{u,sc}}{3} \tag{6-7}$$

using an SF $= 3$ (actually a little over 6, based on the unconfined compression strength). The allowable side shear or skin resistance is computed as

$$f_{s,sc} = \frac{q_{sc}}{30}\lambda_1 \tag{6-7a}$$

where $\lambda_1$ is as follows:

| **Clay soil** | **Sandy soil** | $\lambda_1$ |
|---|---|---|
| $q_u < 20$ kPa | $N_{55} < 5$ | 0.25 |
| $> 20$ kPa | $\geq 5$ | 0.75 |

Point bearing capacity is computed as in Chap. 4 or Chap. 16. Settlements may be computed based on methods given in Chap. 5 or in Chap. 16, and group stresses may be estimated using the methods shown in Fig. 18-4.

Reinforcing bars can be inserted into the fresh SCP if it is necessary to attach a footing or mat securely to the pile or pile group or if the pile(s) must resist bending.

The SCP is particularly suited to anchor floor slabs of dwellings and other buildings in areas where there is a high GWT, possibility of wind shifting the structure or of wave action eroding the soil from beneath the slab. It is also suited for use as an alternative to sand or stone columns if drainage is not a consideration. It may also be used in intermediate locations with sand or stone columns.

## 6-10 JET GROUTING

This procedure is now (1995) being used somewhat in the United States but it has been used elsewhere since the early 1970s. There are several variations on this method. One procedure consists in using a special drill bit with vertical and horizontal high-pressure water jets to excavate through the soil. Cement based grout is then forced through the lateral jets to mix with the small remaining amount of foundation material loosened during excavation. When the grout sets the end result is a fairly hard, impervious column. Clearly this procedure is somewhat similar to the soil-cement columns described earlier.

There are at least four procedures for producing jet-grouted columns, but the two principal methods are

1. Breaking up the soil and mixing it in situ with the grout. A borehole of about the same diameter as the grout rods is used and grout columns up to about 1 m in diameter can be produced.
2. Breaking up and partially removing the in situ material—usually using boreholes much larger than the grout rods—so that the resulting column is mostly grout. Grout columns up to about 3 m in diameter can be produced by this method.

The grout columns (also called *grout piles*) have been used considerably in underpinning structures to provide additional foundation support. The method is also used for general foundation improvement, and very small diameter shafts are sometimes called *root piles*. Closely spaced columns are sometimes used for excavation support (but would require the insertion of reinforcing rods in the wet grout for bending resistance) and for groundwater control; however, the soil-cement columns previously described are probably better suited in most cases. A more comprehensive description of this method is given in ASCE SP 12 [see ASCE (1987)].

## 6-11 FOUNDATION GROUTING AND CHEMICAL STABILIZATION

In addition to the previously described uses of grouting, this term also describes the several techniques of inserting some kind of stabilizing agent into the soil mass under pressure. The pressure forces the agent into the soil voids in a limited space around the injection tube. The agent reacts with the soil and/or itself to form a stable mass. The most common grout is a mixture of cement[3] and water, with or without fine sand.

In general, although grouting is one of the most expensive methods of treating a soil, it has application in

1. Control of water problems by filling cracks and pores; that is, produce a reduction in permeability
2. Prevention of sand densification beneath adjacent structures due to pile driving
3. Reducing both pile driving and operating machinery vibrations by stiffening the soil

Generally this type of grouting can be used if the permeability of the deposit is greater than $10^{-5}$ m/s. One of the principal precautions with grouting is that the injection pressure should not cause the ground surface to heave. In using compaction grouting where a very stiff displacement volume is injected into the ground under high pressure, however, lifting of the ground surface as a grout lens forms is of minor consequence.

[3]Strictly, cement is a complex chemical agent.

Various chemicals can be used as grouting and/or stabilizing agents. Most chemical agents are very expensive for use in foundation treatment. Many, however, have offsetting advantages where low viscosity and setting time must be controlled. An in-depth discussion of the advantages, disadvantages, and availability of chemical stabilizing agents other than those previously described is beyond the scope of this text. The reader is referred to ASCE (1957, 1966) for very extensive bibliographies by the ASCE Committee on Grouting. A more current status report is given by ASCE (1987, pp. 121–135). The following materials are widely used as grout in soil stabilization for road and street work:

Lime

Cement

Fly ash (refer to *Fly Ash: A Highway Construction Material*, U.S. Department of Transportation, June 1976)

Combinations of the above

They can also be used for building construction to improve the soil. Lime, for example, will reduce the plasticity of most clays (by an ion exchange mechanism, usually Ca for Na), which in return reduces volume-change potential (Secs. 7-1 and 7-9).

## 6-12 VIBRATORY METHODS TO INCREASE SOIL DENSITY

The allowable bearing capacity of sands depends heavily on the soil conditions. This is reflected in the penetration number or cone resistance value as well as in the angle of internal friction $\phi$. It is usually not practical to place a footing on loose sand because the allowable bearing capacity (based on settlements) will be too low to be economical.

Additionally, in *earthquake* analyses the local building code may not allow construction unless the relative density is above a certain value. Table 6-1 gives liquefaction-potential relationships between magnitude of earthquake and relative density for a water table about 1.5 m below ground surface. This table can be used for the GWT up to about 3 m below ground surface with slight error. The relative density is related to penetration testing as shown in Table 3-4 after correcting the measured SPT $N$ to $N'_{70}$ [see Eq. (3-3)].

The methods most commonly used to densify cohesionless deposits of sand and gravel with not over 20 percent silt or 10 percent clay are vibroflotation and insertion and withdrawal of a vibrating pile [termed Terra-Probing, see Janes (1973)].

Vibroflotation (patented by the Vibroflotation Foundation Co.) utilizes a cylindrical penetrator 432 mm in diameter, 1.83 m long, and weighing about 17.8 kN. An eccentric mass rotates inside the cylinder at about 1,800 rpm to develop a horizontal centrifugal vibration force of about 90 kN. The device has water jets top and bottom with a flow rate of between 225 and 300 L/min at a pressure of 430 to 580 kPa. Figure 6-6 illustrates the general procedure for using vibroflotation to densify a granular soil mass. The device sinks at a rate of between 1 and 2 m/min into the ground into the "quick" zone under the point caused by a combination of excess water from the lower water jet and vibration. When the Vibroflot reaches the desired depth, depending on footing size and stratum thickness, say 2 to $3B$, and after a few moments of operation, the top jet is turned on and the Vibroflot is withdrawn at the rate of about 0.3 m/min. Sand is added to the crater formed at the top from densification as the device is withdrawn, typically about 10 percent of the compacted volume. Compaction volumes of 7500 to 15 000 $m^3$ in an 8-hr work shift are common. The probe is inserted in a

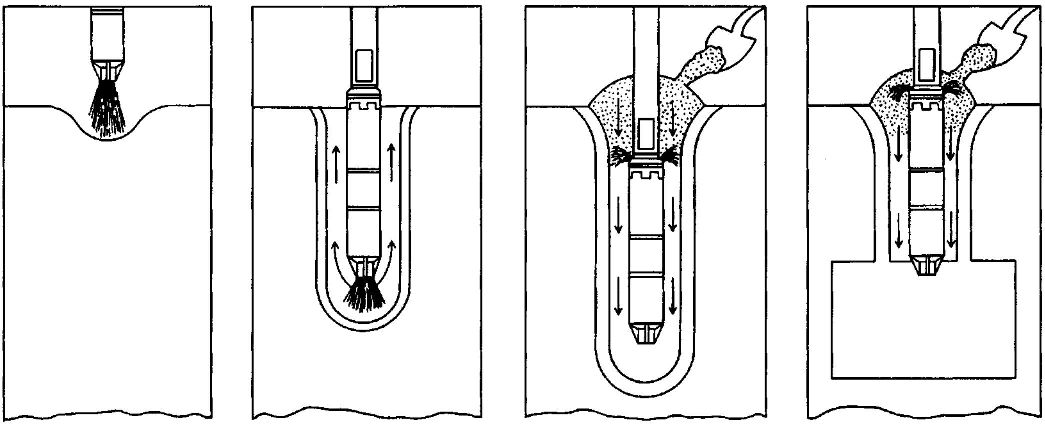
**TABLE 6-1**

## Approximate relationship between earthquake magnitude, relative density, and liquefaction potential for water table 1.5 m below ground surface*

| Earthquake acceleration | High liquefaction probability | Potential for liquefaction depends on soil type and earthquake acceleration | Low liquefaction probability |
|---|---|---|---|
| $0.10g$ | $D_r < 33\%$ | $33 < D_r \le 54$ | $D_r > 54\%$ |
| $0.15g$ | $< 48$ | $48 < D_r \le 73$ | $> 73$ |
| $0.20g$ | $< 60$ | $60 < D_r \le 85$ | $> 85$ |
| $0.25g$ | $< 70$ | $70 < D_r \le 92$ | $> 92$ |

*From Seed and Idriss (1971).

**Figure 6-6** Vibroflotation.



(*a*) Vibroflot is positioned over spot to be compacted, and its lower jet is then opened full.

(*b*) Water is pumped in faster than it can drain away into the subsoil. This creates a momentary "quick" condition beneath the jet which permits the Vibroflot to settle of its own weight and vibration. On typical sites the Vibroflot can penetrate 4.5 to 7.6 m in approximately 2 min.

(*c*) Water is switched from the lower to the top jets, and the pressure is reduced enough to allow water to be returned to the surface, eliminating any arching of backfill material and facilitating the continuous feed of backfill.

(*d*) Compaction takes place during the 0.3 m per minute lifts that return the Vibroflot to the surface. First, the vibrator is allowed to operate at the bottom of the crater. As the sand particles densify, they assume their most compact state. By raising the vibrator step by step and simultaneously back-filling with sand, the entire depth of soil is compacted into a hard core.

grid on 1- to 3- or 5-m centers depending on densification desired, maximum densification being in the immediate vicinity of the probe hole. Bearing capacities of 250 to 400 kPa can be obtained using this method.

The Terra-Probe (patented by the L. B. Foster Co.) method involves mounting a vibratory pile driver on a probe (pile) and vibrating it into and out of the soil to be densified. This device can be used in all soils where the vibroflotation method is applicable. This device is also applicable in underwater work, e.g., shoreline construction. The probe is inserted on spacings of 1.2 to 5 m depending on the amount of densification required.

Whether densification is adequate is determined by comparing in situ $N$ or CPT data before and after vibration. Generally it is necessary to field-test a grid to determine optimum spacing, depth, and any other factors that might affect the efficiency of the process.

## 6-13 USE OF GEOTEXTILES TO IMPROVE SOIL

A *geotextile* (also geofabric) may be defined as a synthetic fabric that is sufficiently durable to last a reasonable length of time in the hostile soil environment. A number of synthetic fabrics made from polyester, nylon, polyethylene, and polypropylene are used to improve the soil in some manner. The fabrics may be woven or knitted into *sheets* and used in either sheets or strips or formed into *geogrids*[4] to reinforce the soil mass. They may be made impermeable for use as waste pond or sanitary landfill liners.

They may be permeable sheets or rods used to drain the soil. For drainage the geotextile depends on having a much larger coefficient of permeability $k$ than the surrounding soil so that the geotextile attracts water by producing a hydraulic gradient between the textile and the soil to be dewatered. For example, placing a permeable fabric against the back of a retaining wall will reduce the lateral (hydrostatic) pressure against the wall as the water intersects the fabric, drains down to drain pipes or holes in the wall, and exits. Permeable rods (wick drains) can be inserted into the soil mass, on spacings somewhat similar to sand drains but much more rapidly, to increase drainage. The water flows laterally to the drain and easily upward to the ground surface since the $k$ of the drain is several orders of magnitude larger than that of the soil being drained. This type of drainage can be used in conjunction with surcharging (similar to sand drains). Certain fabric sheets may be installed in the soil in lieu of sand blankets for soil drainage.

Much of the present use of geotextiles is involved with soil protection or reinforcement. The former involves control of erosion but may also entail isolating a soil mass from water. A particular installation may include excavating 0.5 to 1.5 m of soil that is susceptible to volume change, installing a plastic film, then carefully backfilling. Subsurface water migrating to the surface is blocked by the film so that the upper soil does not become saturated and undergo volume change. Obviously, careful site grading and protection against water entry from above are also required. A similar installation in colder regions can be used to control frost heave. A film of plastic may be used beneath 100 to 150 mm of coarse granular base beneath basement slabs to control basement dampness.

Geotextiles can be used in strips (or sheets or geogrids) to reinforce a soil mass. This usage is common for reinforced earth walls considered in Chap. 12 but may be carried out

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[4]A geotextile grid is a section of specified dimensions consisting of bars of some size intersecting at right angles. Grids are similar to welded wire fabric except that usually the grid rods in one direction do not lie on top of the rods in the orthogonal direction.

for embankments so that steeper slopes can be used or so that compaction can be made to the edge of the slope, or to improve the bearing capacity of poor soil underlying the embankment.

Geotextiles and geogrids have potential application beneath footings and across culverts, both to improve bearing capacity and to spread the loaded area. The interaction of the fabric (dimensions large relative to soil grains) and soil effectively increases the angle of internal friction (between fabric and soil) and cohesion (fabric tension). Current problems with using geotextile sheets/strips or geogrids to increase bearing capacity are in determining the horizontal and vertical spacings of the reinforcement and in controlling settlement. Since improvement is being made on poor ground, the reinforcement will carry substantial tensile stresses. Geotextiles in tension tend to deform considerably (they stretch) under relatively small stresses. Foundation reinforcements would, as a consequence, have to be relatively thick in order to control vertical movement—and thickness is directly related to cost. Alternatives such as piles or soil excavation and replacement with imported fill may be more economical than excavation and replacement with existing soil and geotextile reinforcement.

At the present time, an abundance of theory is not available to compute the required amount, type, or geometry of geotextile reinforcement.

For hazardous fill and similar lining applications strength is not the major consideration, but great care must be exercised to ensure that sheet laps are sealed so that contaminated leachate cannot escape. It is necessary to lap and seal sheets since liners may cover several hectares (or acres) of ground and sheets are available in finite widths usually under about 5 m.

Geotextiles have not yet been in use for a long service period, but their use is spreading very rapidly. There have been, to date, several international conferences on geotextile usage, a textbook by Koerner (1990), occasional papers in the several applicable journals cited in this text, the ASCE (1987) special publication, as well as a Geotextile Fabrics Report.[5] There are also regularly scheduled international conferences on geotextiles.

## 6-14 ALTERING GROUNDWATER CONDITIONS

From the concept of submerged unit weight it is evident that the intergranular pressure can be increased by removing the buoyant effect of water. This can be accomplished by lowering the water table. In many cases this may not be feasible or perhaps only as a temporary expediency. Where it is possible, one obtains the immediate increase in intergranular pressure of $\gamma_w z_w$, where $z_w$ is the change in GWT elevation.

It is usually impossible to lower the GWT exactly within the limits of one's own property. Thus, the increase in effective pressure also occurs beneath adjacent properties and can result in damage to those owners. The result may be cracked pavements and/or buildings, and the owners will certainly seek damages.

Note that it may be possible to raise the GWT. This process can also have an adverse effect on adjacent properties and requires careful analysis before being undertaken.

Since any activity that alters the GWT location will have some kind of effect on the environment, it will usually be necessary to get permission from appropriate environmental agencies. Otherwise litigation is almost certain to follow.

[5]Published by the Industrial Fabrics Association International, 345 Cedar Street, Suite 800, St. Paul, MN 55101 [Tel.: (612)-222-2508]. This monthly magazine usually describes one or more geotextile applications. A yearly summary volume containing a list of manufacturers, geotextile products available, and selected engineering data on the several products such as strength, deformation characteristics, sheet widths, etc., is also published.

# PROBLEMS

**6-1.** The penetration number $N$ of a loose sand varies from 7 at elevation $-1.5$ m to 16 at elevation $-7.0$ m. It is necessary to have a $D_r$ of at least 0.75 for this soil. The area to be covered is $40 \times 50$ m. Vibroflotation or Terra-Probing will be used. What will be the expected $N'_{70}$ values after densification? About how many cubic meters of sand will be required to maintain the existing ground elevation? (*Note:* Your answer depends on your assumptions.)

**6-2.** What is the additional settlement due to lowering the water table of Example 5-14 from 349.5 to 344.0? Comment on the effect of raising the water table to elevation 354.5 ft.

**6-3.** Compute the zero-air-voids curve for soil no. 2 of Fig. 6-1 using $G_s = 2.65$ and plot it on a copy of the figure (or an overlay that shows the compaction curves together with the ZAV curve). Is this $G_s$ reasonably correct for this soil? If not what would you use for $G_s$?

**6-4.** A soft clay deposit with $s_u = 20$ kPa (from $q_u$ tests) is 8.0 m thick and is underlain by a dense sandy gravel. The site is to be used for oil storage tanks. The water table is approximately at ground surface. The area is $400 \times 550$ m. Other soil data include the following:

$$k_h = 4 \times 10^{-6}\ \text{m/s} \qquad w_L = 62\% \qquad w_P = 31\% \qquad w_N = 58\%$$
$$G_s = 2.63 \qquad c_v = 8.64 \times 10^{-4}\ \text{m}^2/\text{day}$$

Describe how you would prepare this site for use. How would you either remove 700 mm of anticipated settlement in the clay prior to installing the storage tanks or otherwise control settlement? The tank pressure loading including tank and oil is 110 kPa. The tank has a diameter of 10 m, and it is desirable that the tanks not settle over 25 mm additional from the preload position when filled.

**6-5.** In referring to Sec. 6-5.1, sand or wick drains are spaced on 3-m centers in a clay soil. Tests indicate the vertical $c_v = 1 \times 10^{-3}$ m²/day and the horizontal value $= 4c_v$. Estimate how long it will take for a 3-m depth of this clay to undergo 80 percent consolidation.
*Answer:* $\approx$ 0.87 years

**6-6.** What drain spacing in Problem 6-5 would be required to reduce the consolidation time to 0.5 years (6 months)?

**6-7.** Redo Example 6-1 with a final void ratio $e_f = 0.45$ (instead of 0.5), and estimate the volume of sand required if the sand columns have a depth of 3 m.
*Answer:* 5.25 m³(for each $3 \times 3$ m grid)

**6-8.** For a stone column we have $\phi' = 42°$ and a clay cohesion $c = 1$ kPa. For a SF $= 2$, what might the allowable bearing pressure be using Eq. (6-5)? Hint: Assume a diameter $D_{col}$ and length $L_c$.

**6-9.** A stone column is installed in a soft clay. The drill diameter $= 800$ mm and the shaft depth $L_c = 3.5$ m. If the volume of stone used to construct the column $V_c = 2.8$ m³, what is the nominal column diameter $D_c$?
*Answer:* $\approx$ 1.0 m

**6-10.** A 2.5-m diameter stone column is installed in a clay soil with $c_s = 1.1$ and $c_p = 0.8$ kPa. If the ultimate load $P_{ult} = 90$ kN and a SF $= 1.5$ is used, what is the required column depth $L_c$? *Hint:* The working load $P_w = P_{ult/SF}$.
*Answer:* $\approx$ 6.0 m

**6-11.** A 3-m length of geotextile fabric is installed in a pull-out (tension) condition. The soil has a $\phi = 34°$, and the vertical pressure on the strip is 25 kPa. The coefficient of friction $f = \tan\phi$.
What is the approximate pull-out force on the fabric strip if it is 100 mm wide? *Hint:* Friction acts on both the top and bottom of the strip.
*Answer:* $\approx$ 10.1 kN