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Methods of Soft Ground Improvement

James D. Hussin

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12.1 Introduction

When a suitable foundation has to be designed for a superstructure, the foundation engineer typically follows a decision-making process in selecting the optimum type of foundation. The flowchart shown in Figure 12.1 illustrates the important steps of that decision process, which is based on the principle that cost-effective alternatives must be sought first before considering relatively costly foundation alternatives. It is seen that, in keeping with the decision sequence advocated in Figure 12.1, one must consider applicable site specific techniques for improvement of soft ground conditions, before resorting to deep foundations.

This chapter gives an overview of techniques that are commonly used by specialty contractors in the United States to improve the performance of the ground *in situ*. Not included are less specialized methods of ground improvement such as surface compaction with vibratory rollers or sheep foot type compactors, or methods that involve the placement of geotextile or geogrid materials in soil fill as it is placed. The techniques are divided into three categories:

1. *Compaction* — techniques that typically are used to compact or densify soil *in situ*.
2. *Reinforcement* — techniques that typically construct a reinforcing element within the soil mass without necessarily changing the soil properties. The performance of the soil mass is improved by the inclusion of the reinforcing elements.
3. *Fixation* — techniques that fix or bind the soil particles together thereby increasing the soil's strength and decreasing its compressibility and permeability.

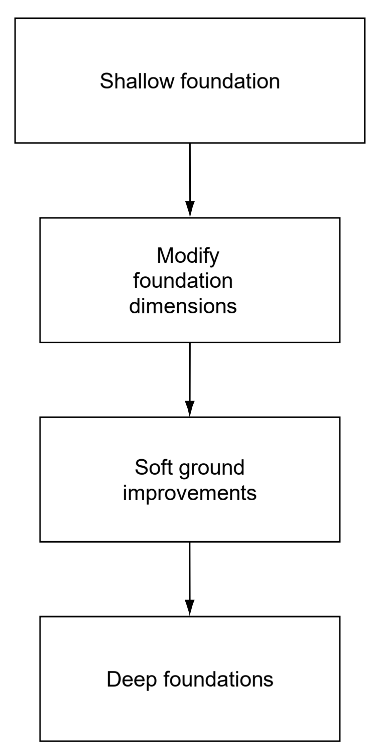


FIGURE 12.1
Decision process involved with selection of foundation type.

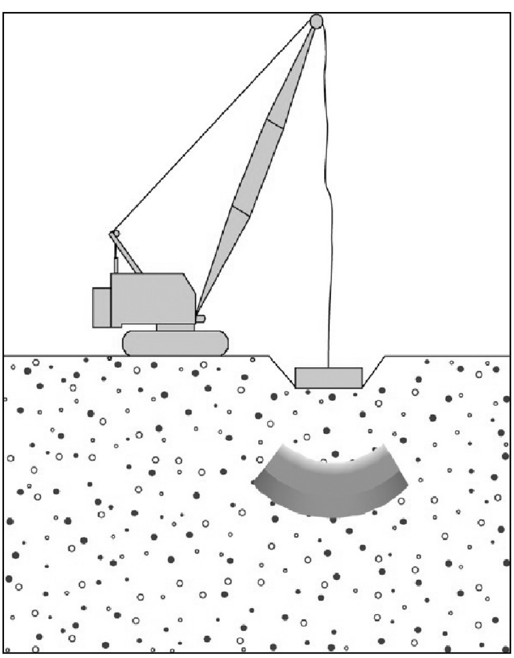
Techniques have been placed in the category in which they are most commonly used even though several of the techniques could fall into more than one of the categories. As each technique is addressed, the expected performance in different soil types is presented. An overview of the design methodology for each technique is also presented as are methods of performing quality assurance and quality control (QA/QC). Several *in situ* techniques of soil improvement exist that are not commonly used. These techniques are briefly described at the end of each category.

This chapter is intended to give the reader a general understanding of each of the techniques, how each improves the soil performance, and an overview of how each is analyzed. The purpose is neither to present all the nuances of each technique nor to be a detailed design manual. Indeed, entire books have been written on each technique separately. In addition, this chapter does not address all the safety issues associated with each technique. Many of these techniques have inherent dangers associated with them and should only be performed by trained and experienced specialty contractors with documented safety records.

12.2 Compaction

12.2.1 Dynamic Compaction

Dynamic compaction (DC), also known as dynamic deep compaction, was advanced in the mid-1960s by Luis Menard, although there are reports of the procedure being performed over 1000 years ago. The process involves dropping a heavy weight on the surface of the ground to compact soils to depths as great as 40 ft or 12.5 m (Figure 12.2). The



(a)

(b)

FIGURE 12.2
Deep dynamic compaction: (a) schematic, (b) field implementation. (From Hayward Baker Inc. With permission.)

method is used to reduce foundation settlements, reduce seismic subsidence and liquefaction potential, permit construction on fills, densify garbage dumps, improve mine spoils, and reduce settlements in collapsible soils.

Applicable soil types: Dynamic compaction is most effective in permeable, granular soils. Cohesive soils tend to absorb the energy and limit the technique's effectiveness. The expected improvement achieved in specific soil types is shown in Table 12.1. The ground water table should be at least 6 ft below the working surface for the process to be effective. In organic soils, dynamic compaction has been used to construct sand or stone columns by repeatedly filling the crater with sand or stone and driving the column through the organic layer.

Equipment: Typically a cycle duty crane is used to drop the weight, although specially built rigs have been constructed. Since standard cranes are typically not designed for the high cycle, dynamic loading, the cranes must be in good condition and carefully maintained and inspected during performance of the work to maintain a safe working environment. The crane is typically rigged with sufficient boom to drop the weight from heights of 50 to 100 ft (15.4 to 30.8 m), with a single line to allow the weight to nearly ''free fall,'' maximizing the energy of the weight striking the ground. The weight to be dropped must be below the safe single line capacity of the crane and cable. Typically weights range from 10 to 30 tons (90 to 270 kN) and are constructed of steel to withstand the repetitive dynamic forces.

Procedure: The procedure involves repetitively lifting and dropping a weight on the ground surface. The layout of the primary drop locations is typically on a 10 to 20 ft (3.1 to 6.2 m) grid with a secondary pass located at the midpoints of the primary pass. Once the crater depth has reached about 3 to 4 ft (about 1 m), the crater is filled with granular material before additional drops are performed at that location.

The process produces large vibrations in the soil which can have adverse effects on nearby existing structures. It is important to review the nearby adjacent facilities for vibration sensitivity and to document their preexisting condition, especially structures within 500 ft (154 m) of planned drop locations. Vibration monitoring during DC is also prudent. Extreme care and careful monitoring should be used if treatment is planned within 200 ft (61.5 m) of an existing structure.

Materials: The craters resulting from the procedure are typically filled with a clean, free draining granular soil. A sand backfill can be used when treating sandy soils. A crushed stone backfill is typically used when treating finer-grained soils or landfills.

TABLE 12.1

Expected Improvement and Required Energy with Dynamic Compaction

Soil Description	Expected Improvement	Typical Energy Required (tons ft/cf)[a]
Gravel and sand <10% silt, no clay	Excellent	2–2.5
Sand with 10–80% silt and <20% clay, pI < 8	Moderate if dry; minimal if moist	2.5–3.5
Finer-grained soil with pI > 8	Not applicable	—
Landfill	Excellent	6–11

[a]Energy ¼ (drop height weight number of drops)/soil volume to be compacted; 1 ton ft/ft^3 ¼ 94.1 kJ/m^3.

Design: The design will begin with an analysis of the planned construction with the existing subsurface conditions (bearing capacity, settlement, liquefaction, etc.). Then the same analysis is performed with the improved soil parameters (i.e., SPT N value, etc.) to determine the minimum values necessary to provide the required performance. Finally, the vertical and lateral extent of improved soil necessary to provide the required performance is determined.

The depth of influence is related to the square root of the energy from a single drop (weight times the height of the drop) applied to the ground surface. The following correlation was developed by Dr Robert Lucas based on field data:

$$D \text{ ¼ } k(W \quad H)^{1=2} \qquad (12:1)$$

where D is the maximum influence depth in meters beneath the ground surface, W is the weight in metric tons (9 kN) of the object being dropped, and H is the drop height in meters above the ground surface. The constant k varies with soil type and is between 0.3 and 0.7, with lower values for finer-grained soils.

Although this formula predicts the maximum depth of improvement, the majority of the improvement occurs in the upper two-thirds of this depth with the improvement tapering off to zero in the bottom third. Repeated blows at the same location increases the degree of improvement achieved within this zone. However, the amount of improvement achieved decreases with each drop eventually resulting in a point of diminishing returns. The expected range of unit energy required to achieve this point is presented in Table 12.1.

Treatment of landfills is effective in reducing voids; however, it has little effect on future decomposition of biodegradable components. Therefore treatment of landfills is typically restricted to planned roadway and pavement areas, and not for structures. After completion of dynamic compaction, the soils within 3 to 4 ft (1 m) of the surface are loose.

The surface soils are compacted with a low energy ''ironing pass,'' which typically consists of dropping the same weight a couple of times from a height of 10 to 15 ft (3.0 to 4.5 m) over the entire surface area.

Quality control and quality assurance: In most applications, penetration testing is performed to measure the improvement achieved. In landfills or construction debris, penetration testing is difficult and shear wave velocity tests or large scale load tests with fill mounds can be performed. A test area can be treated at the beginning of the program to measure the improvement achieved and to make adjustments if required. The depth of the craters can also be measured to detect ''soft'' areas of the site requiring additional treatment. The decrease in penetration with additional drops gives an indication when sufficient improvement is achieved.

12.2.2 Vibro Compaction

Vibro compaction (VC), also known as Vibroflotation™ was developed in the 1930s in Europe. The process involves the use of a down-hole vibrator (vibroflot), which is lowered into the ground to compact the soils at depth (Figure 12.3). The method is used to increase bearing capacity, reduce foundation settlements, reduce seismic subsidence and liquefaction potential, and permit construction on loose granular fills.

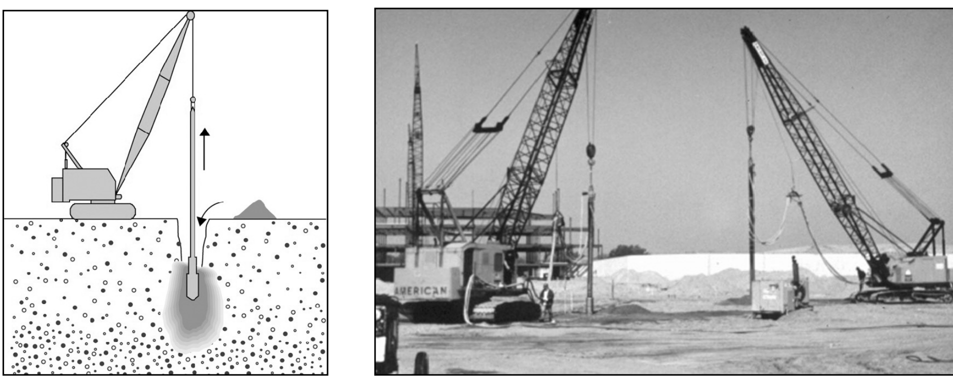
Applicable soil types: The VC process is most effective in free draining granular soils. The expected improvement achieved in specific soil types is shown in Table 12.2. The typical spacing is based on a 165-horsepower (HP) (124 kW) vibrator. Although most effective below the groundwater table, VC is also effective above.

TABLE 12.2

Expected Improvement and Typical Probe Spacing with Vibro Compaction

Soil Description	Expected Improvement	Typical Probe Spacing (ft)[a]
Well-graded sand <5% silt, no clay	Excellent	9–11
Uniform fine to medium sand with <5% silt and no clay	Good	7.5–9
Silty sand with 5–15% silt, no clay	Moderate	6–7.5
Sand/silts, >15% silt	Not applicable[b]	—
Clays and garbage	Not applicable	—

[a]Probe spacing to achieve 70% relative density with 165 HP vibroflot, higher densities require closer spacing (1 ft ¼ 0.308 m).
[b]Limited improvement in silts can be achieved with large displacements and stone backfill.



(a) (b)

FIGURE 12.3
Vibroflotation: (a) schematic, (b) field implementation. (From Hayward Baker Inc. With permission.)

Equipment: The vibroflot consists of a cylindrical steel shell with and an interior electric or hydraulic motor which spins an eccentric weight (Figure 12.4). Common vibrator dimensions are approximately 10 ft (3.1 m) in length and 1.5 ft (0.5 m) in diameter. The vibration is in the horizontal direction and the source is located near the bottom of the probe, maximizing the effect on the surrounding soils. Vibrators vary in power from about 50 to over 300 HP (37.7 to 226 kW). Typically, the vibroflot is hung from a standard crane, although purpose built machines do exist. Extension tubes are bolted to the top of the vibrator so that the vibrator can be lowered to the necessary treatment depth.

Electric vibrators typically have a remote ammeter, which displays the amperage being drawn by the electric motor. The amperage will typically increase as the surrounding soils densify.

Procedure: The vibrator is lowered into the ground, assisted by its weight, vibration, and typically water jets in its tip. If difficult penetration is encountered, predrilling through the firm soils may also be performed. The compaction starts at the bottom of the treatment depth. The vibrator is then either raised at a certain rate or repeatedly raised and lowered as it is extracted (Figure 12.5). The surrounding granular soils rearranged into a denser configuration, achieving relative densities of 70 to 85%. Treatment as deep as 120 ft (37 m) has been performed.

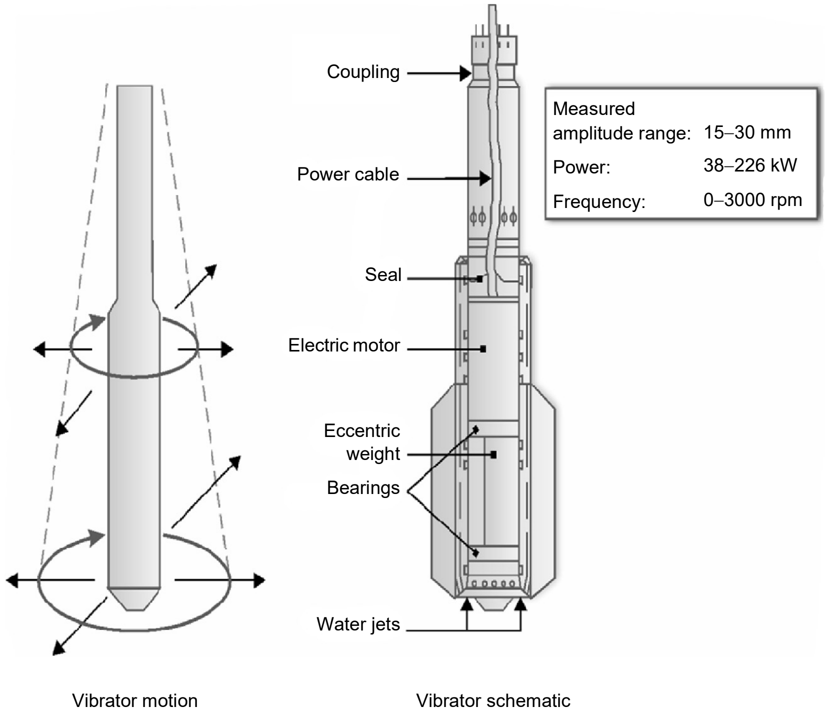


FIGURE 12.4
Electric vibroflot cross section. (From Hayward Baker Inc. With permission.)

Sand added around the vibrator at the ground surface falls around the vibrator to its tip to compensate for the volume reduction during densification. If no sand is added, the *in situ* sands will fall, resulting in a depression at the ground surface. Loose sand will experience a 5 to 15% volume reduction during densification. Coarser backfill, up to gravel size, improves the effectiveness of the technique, especially in silty soils. The technique does not densify the sands within 2 to 3 ft (0.6 to 0.9 m) of the ground surface. If necessary, this is accomplished with a steel drum vibratory roller.

Materials: Backfill usually consists of sand with less than 10% silt and no clay, although gravel size backfill can also be used. A coarser backfill facilitates production and densification.

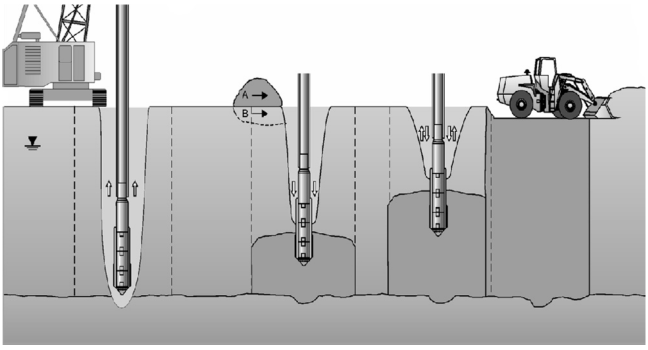


FIGURE 12.5
Vibro compaction process. (From Hayward Baker Inc. With Permission.)

Design: The design will begin with an analysis of the planned construction with the existing subsurface conditions (bearing capacity, settlement, liquefaction, etc.). Then the same analysis is performed with the improved soil parameters (i.e., SPT *N* value, etc.) to determine the minimum soil parameters necessary to provide the required performance. And finally, the vertical and lateral extent of improved soil necessary to provide the required performance is determined. In the case of settlement improvement for spread footings, it is common to improve the sands beneath the planned footings to a depth of twice the footing width for isolated column footings and four times the footing width for wall footings. Area treatments are required where an area load is planned or in seismic applications. For treatment beneath shallow foundations for nonseismic conditions, it is common to treat only beneath the foundations (Figure 12.6).

The degree of improvement achievable depends on the energy of the vibrator, the spacing of the vibrator penetrations, the amount of time spent densifying the soil, and the quantity of backfill added (or *in situ* soil volume reduction).

Quality control and quality assurance: Production parameters should be documented for each probe location, such as depth, compaction time, amperage increases, and estimated volume of backfill added. If no backfill is added, the reduction in the ground surface elevation should be recorded. The degree of improvement achieved is typically measured with penetration tests performed at the midpoint of the probe pattern.

12.2.3 Compaction Grouting

Compaction grouting, one of the few US born ground improvement techniques, was developed by Ed Graf and Jim Warner in California in the 1950s. This technique densifies soils by the injection of a low mobility, low slump mortar grout. The grout bulb expands as additional grout is injected, compacting the surrounding soils through compression. Besides the improvement in the surrounding soils, the soil mass is reinforced by the resulting grout column, further reducing settlement and increasing shear strength. The method is used to reduce foundation settlements, reduce seismic subsidence and liquefaction potential, permit construction on loose granular fills, reduce settlements in collapsible soils, and reduce sinkhole potential or stabilize existing sinkholes in karst regions.

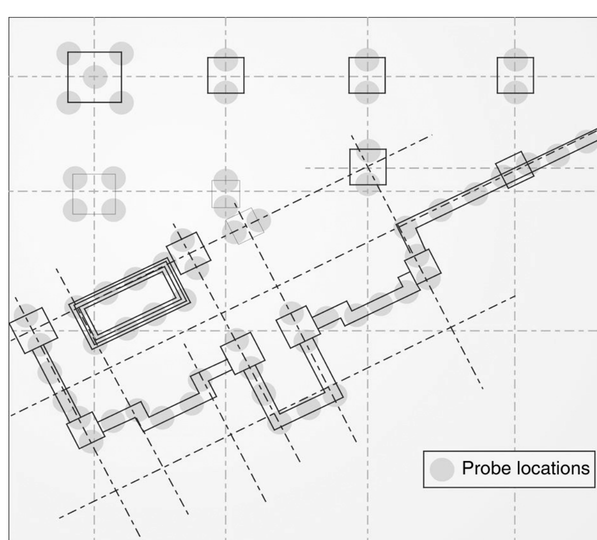


FIGURE 12.6
Typical vibro compaction layout for nonseismic treatment beneath foundations. (From Hayward Baker Inc. With permission.)

Applicable soil types: Compaction grouting is most effective in free draining granular soils and low sensitivity soils. The expected improvement achieved in specific soil types is shown in Table 12.3. The depth of the groundwater table is not important as long as the soils are free draining.

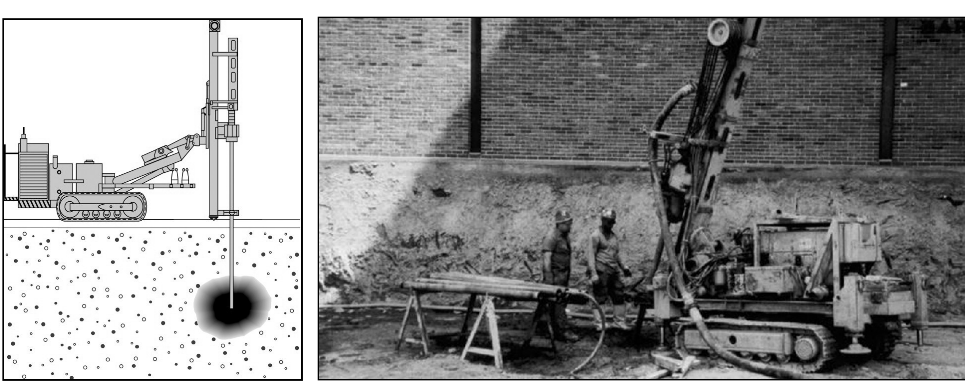
Equipment: Three primary pieces of equipment are required to perform compaction grouting, one to batch the grout, one to pump the grout, and one to install the injection pipe. In some applications, ready-mix grout is used eliminating the need for on-site batching. The injection pipe is typically installed with a drill rig or is driven into the ground. It is important that the injection pipe is in tight contact with the surrounding soils. Otherwise the grout might either flow around the pipe to the ground surface or the grout pressure might jack the pipe out of the ground. Augering or excessive flushing could result in a loose fit. The pump must be capable of injecting a low slump mortar grout under high pressure. A piston pump capable of achieving a pumping pressure of up to 1000 psi (6.9 MPa) is often required (Figure 12.7).

Procedure: Compaction grouting is typically started at the bottom of the zone to be treated and precedes upward (Figure 12.8). The treatment does not have to be continued to the ground surface and can be terminated at any depth. The technique is very effective in targeting isolated zones at depth. It is generally difficult to achieve significant improvement within about 8 ft (2.5 m) of the ground surface. Some shallow improvement can be accomplished using the slower and more costly top down procedure. In this procedure, grout is first pumped at the top of the treatment zone. After the grout sets up, the pipe is

TABLE 12.3

Expected Improvement with Compaction Grouting

Soil Description	Densification	Reinforcement
Gravel and sand <10% silt, no clay	Excellent	Very good
Sand with between 10 and 20% silt and <2% clay	Moderate	Very good
Finer-grained soil, nonplastic	Minimal	Excellent
Plastic soil	Not applicable	Excellent



(a) (b)

FIGURE 12.7
Compaction grout process: (a) schematic, (b) field implementation. (From Hayward Baker Inc. With permission.)

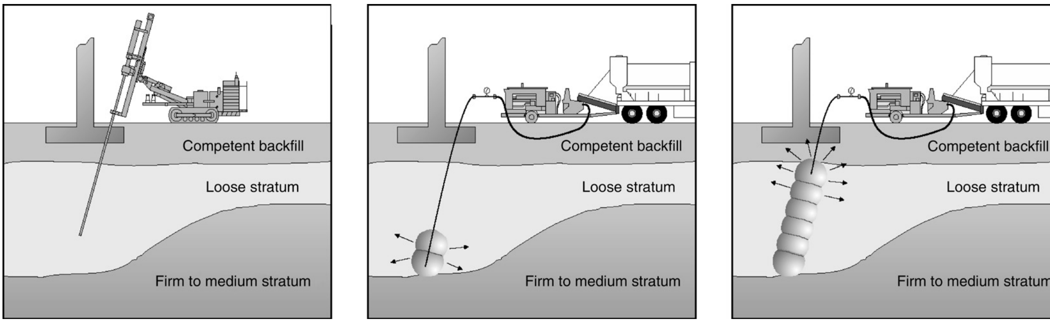


FIGURE 12.8
Compaction grouting process. (From Hayward Baker Inc. With permission.)

drilled to the underside of the grout and additional grout is injected. This procedure is repeated until the bottom of the treatment zone is grouted. The grout injection rate is generally in the range of 3 to 6 ft^3/min (0.087 to 0.175 m^3/min), depending on the soils being treated. If the injection rate is too fast, excess pore pressures or fracturing of the soil can occur, reducing the effectiveness of the process.

Materials: Generally, the compaction grout consists of Portland cement, sand, and water. Additional fine-grained materials can be added to the mix, such as natural fine-grained soils, fly ash, or bentonite (in small quantities). The grout strength is generally not critical for soil improvement, and if this is the case, cement has been omitted and the sand replaced with naturally occurring silty sand. A minimum strength may be required if the grout columns or mass are designed to carry a load.

Design: The design will begin with an analysis of the planned construction with the existing subsurface conditions (bearing capacity, settlement, liquefaction, etc.). Then the same analysis is performed with the improved soil parameters (i.e., SPT N value, etc.) to determine the minimum parameters necessary to provide the required performance. Finally, the vertical and lateral extent of improved soil necessary to provide the required performance is determined. In the case of settlement improvement for spread footings, it is common to improve the sands beneath the planned footings to a depth of twice the footing width for isolated column footings and four times the footing width for wall footings. A conservative analysis of the post-treatment performance only considers the improved soil and does not take into account the grout elements. The grout elements are typically columns. A simplified method of accounting for the grout columns is to take a weighted average of the parameters of the improved soil and grout. The grout columns can also be designed using a standard displacement pile methodology.

The degree of improvement achievable depends on the soil (soil gradation, percent fines, percent clay fines, and moisture content) as well as the spacing and percent displacement (the volume of grout injected divided by volume of soil being treated).

Quality control and quality assurance: Depending on the grout requirements, grout slump and strength is often specified. Slump testing and sampling for unconfined compressive strength testing is performed during production. The production parameters should also be monitored and documented, such as pumping rate, quantities, pressures, ground heave, and injection depths. Postgrouting penetration testing can be performed between injection locations to verify the improvement of granular soil.

12.2.4 Surcharging with Prefabricated Vertical Drains

Surcharging consists of placing a temporary load (generally soil fill) on sites to pre-consolidate the soil prior to constructing the planned structure (Figure 12.9). The process improves the soil by compressing the soil, increasing its stiffness and shear strength. In partially or fully saturated soils, prefabricated vertical drains (PVDs) can be placed prior to surcharge placement to accelerate the drainage, reducing the required surcharge time.

Applicable soil types: Preloading is best suited for soft, fine-grained soils. Soft soils are generally easy to penetrate with PVDs and layers of stiff soil may require predrilling.

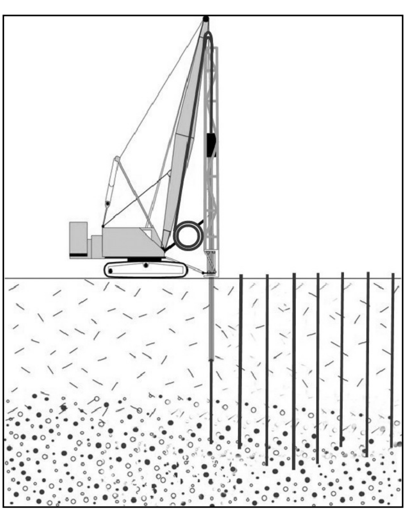
Equipment: Generally, a surcharge consists of a soil embankment and is placed with standard earthmoving equipment (trucks, dozers, etc). Often the site surface is soft and wet, requiring low ground pressure equipment.

The PVDs are installed with a mast mounted on a backhoe or crane, often with low ground pressure tracks. A predrilling rig may be required if stiff layers must be penetrated.

Procedure: Fill soil is typically delivered to the area to be surcharged with dump trucks. Dozers are then used to push the soil into a mound. The height of the mound depends on the required pressure to achieve the required improvement.

The PVDs typically are in 1000 ft (308 m) rolls and are fed into a steel rectangular tube (mandrel) from the top. The mandrel is pushed, vibrated, driven or jetted vertically into the ground with a mast mounted on a backhoe or crane. An anchor plate or bar attached to the bottom of the PVD holds it in place in the soil as the mandrel is extracted. The PVD is then cut off slightly above the ground surface and another anchor is attached. The mandrel is moved to the next location and the process is repeated. If obstructions are encountered during installation, the wick drain location can be slightly offset.

In very soft sites, piezometers and inclinometers, as well as staged loading, may be required to avoid the fill being placed too quickly, causing a bearing capacity or slope stability failure. If stiff layers must be penetrated, predrilling may be required.



(a)

(b)

FIGURE 12.9
Surcharging with prefabricated vertical drains: (a) schematic, (b) field implementation. (From Hayward Baker Inc. With permission.)

Settlement plates are placed in the surcharge. The elevation of these plates is measured to determine when the design settlement has occurred.

Materials: The first layer of surcharge generally consists of a drainage material to drain the water displaced from the ground during compression. Since surcharge soils are generally temporary in nature, their composition and degree of compaction are generally not critical. If the site settlement will result in some of the surcharge soil settling below finish grade, this height of fill is initially placed as compacted structural fill, to avoid having to excavate and replace it at the end of the surcharge program.

The PVD is composed of a 4-in. (10 cm) wide strip of corrugated or knobbed plastic wrapped in a woven filter fabric. The fabric is designed to remain permeable to allow the ground water to flow through it but not the soil.

Design: Generally, a surcharge program is considered when the site is underlain by soft fine-grained soils which will experience excessive settlement under the load of the planned structure. Using consolidation test data, a surcharge load and duration is selected to preconsolidate the soils sufficiently such that when the surcharge load is removed and the planned structure is constructed, the remaining settlement is acceptable.

PVDs are selected if the required surcharge time is excessive for the project. The time required for the surcharge settlement to occur depends on the time it takes for the excess pore water pressure to dissipate. This is dictated by the soils permeability and the square of the distance the water has to travel to get to a permeable layer. The PVDs accelerate the drainage by shortening the drainage distance. The spacing of the PVDs are designed to reduce the consolidation time to an acceptable duration. The closer the drains are installed (typically 3 to 6 ft on center) the shorter the surcharge program is in duration.

Quality control and quality assurance: The height and unit weight of the surcharge should be documented to assure that the design pressure is being applied. The PVD manufacturer's specifications should be reviewed to confirm that the selected PVD is suitable for the application. During installation, the location, depth, and verticality are important to monitor and record. The settlement monitoring program is critical so that the completion of the surcharge program can be determined.

12.2.5 Infrequently-Used Compaction Techniques

12.2.5.1 Blast-Densification and Vacuum-Induced Consolidation

Blast-densification densifies sands with underground explosives. The technique was first used in the 1930s in the former Soviet Union and in New Hampshire. The below grade explosion causes volumetric strains and shearing which rearranges of soil particles into a denser configuration. The soils are liquefied and then become denser as the pore pressures dissipate. Soils as deep as 130 ft (40 m) have been treated. A limited number of projects have been performed and generally only for remote location where the blast-induced vibrations are not a concern.

Vacuum-induced consolidation (VIC) uses atmospheric pressure to apply a temporary surcharge load. The concept of VIC was introduced in the 1950s; however, the first practical project was performed in 1980 in China. Following that, a number of small projects have been performed, but few outside China. A porous layer of sand or gravel is placed over the site and it is covered with an air tight membrane, sealed into the clay below the ground surface. The air is then pumped out of the porous layer, producing a pressure difference of 0.6 to 0.7 atm, equivalent to about 15 ft (4.6 m) of fill. The process can be accelerated by the use of PVDs. The process eliminates the need for surcharge fill and avoids shear failure in the soft soil; however, any sand seams within the compressible layer can make it difficult to maintain the vacuum.

12.3 Reinforcement

12.3.1 Stone Columns

Stone columns refer to columns of compacted, gravel size stone particles constructed vertically in the ground to improve the performance of soft or loose soils. The stone can be compacted with impact methods, such as with a falling weight or an impact compactor or with a vibroflot, the more common method. The method is used to increase bearing capacity (up to 5 to 10 ksf or 240 to 480 kPa), reduce foundation settlements, improve slope stability, reduce seismic subsidence, reduce lateral spreading and liquefaction potential, permit construction on loose/soft fills, and precollapse sinkholes prior to construction in karst regions.

Applicable soil types: Stone columns improve the performance of soils in two ways, densification of surrounding granular soil and reinforcement of the soil with a stiffer, higher shear strength column. The expected improvement achieved in specific soil types is shown in Table 12.4. The depth of the ground water is generally not critical.

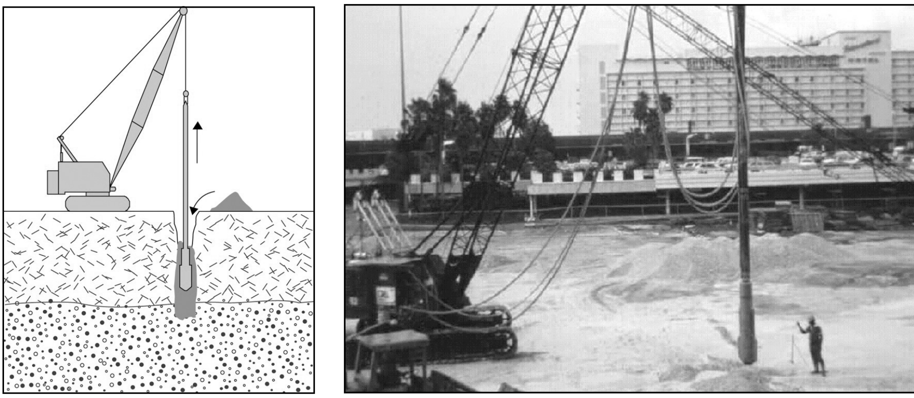
Procedure: The column construction starts at the bottom of the treatment depth and proceeds to the surface. The vibrator penetrates into the ground, assisted by its weight, vibration, and typically water jets in its tip, the wet top feed method (Figure 12.10 and Figure 12.11a). If difficult penetration is encountered, predrilling through the firm soils may also be performed. A front end loader places stone around the vibroflot at the ground surface and the stone falls to the tip of the vibroflot through the flushing water around the exterior of the vibroflot. The vibrator is then raised a couple of feet and the stone falls around the vibroflot to the tip, filling the cavity formed as the vibroflot is raised. The vibroflot is then repeatedly raised and lowered as it is extracted, compacting and displacing the stone in 2 to 3 ft (0.75 to 0.9 m) lifts. The flushing water is usually directed to a settlement pond where the suspended soil fines are allowed to settle.

If the dry bottom feed procedure is selected, the vibroflot penetrates into the ground, assisted by its weight and vibrations alone (Figure 12.11b). Again, predrilling may be used if necessary or desired. The remaining procedure is then similar except that the stone is feed to the tip of the vibroflot though the tremie pipe. Treatment depth as deep as 100 ft (30 m) has been achieved.

TABLE 12.4

Expected Densification and Reinforcement Achieved with Stone Columns

Soil Description	Densification	Reinforcement
Gravel and sand <10% silt, no clay	Excellent	Very good
Sand with between 10 and 20% silt and <2% clay	Very good	Very good
Sand with >20% silt and nonplastic silt	Marginal (with large displacements)	Excellent
Clays	Not applicable	Excellent



(a) (b)

FIGURE 12.10
Installation of stone columns: (a) schematic, (b) field implementation. (From Hayward Baker Inc. With permission.)

Equipment: When jetting water is used to advance the vibroflot, the equipment and setup is similar to VC. If jetting water is not desired for a particular project, the dry bottom feed process can be used (Figure 12.11b). A tremie pipe, through which stone is fed to the tip of the vibroflot, is fastened to the side of the vibroflot. A stone skip is filled with stone on the ground with a front end loader and a separate cable raises the skip to a chamber at the top of the tremie pipe.

A specific application is referred to as vibro piers. The process refers to short, closely spaced stone columns designed to create a stiff block to increase bearing capacity and reduce settlement to acceptable values. Vibro piers are typically constructed in cohesive soils in which a full depth predrill hole will stay open. The stone is compacted in 1 to 2 ft (0.4 to 0.8 m) lifts, each of which is rammed and compacted with the vibroflot.

Materials: The stone is typically a graded crushed hard rock, although natural gravels and pebbles have been used. The greater the friction angle of the stone, the greater the modulus and shear strength of the column.

Design: Several methods of analysis are available. For static analysis, one method consists of calculating weighted averages of the stone column and soil properties (cohesion, friction angle, etc.). The weighted averages are then used in standard geotechnical methods of analysis (bearing capacity, settlement, etc.). Another method developed by Dr Hans Priebe, involves calculating the post-treatment settlement by dividing the untreated settlement by an improvement factor (Figure 12.12). In static applications, the treatment limits are typically equal to the foundation limits.

For liquefaction analysis, stone column benefits include densification of surrounding granular soils, reduction in the cyclic stress in the soil because of the inclusion of the stiffer stone columns, and drainage of the excess pore pressure. A method of evaluation for all three of these benefits was presented by Dr Juan Baez. Dr Priebe has also presented a variation of his static method for this application. In liquefaction applications, the treatment generally covers the structure footprint and extends laterally outside the areas to be protected, a distance equal to two-thirds of the thickness of the liquefiable zone.

This is necessary to avoid surrounding untreated soils from adversely affecting the treated area beneath the foundation.

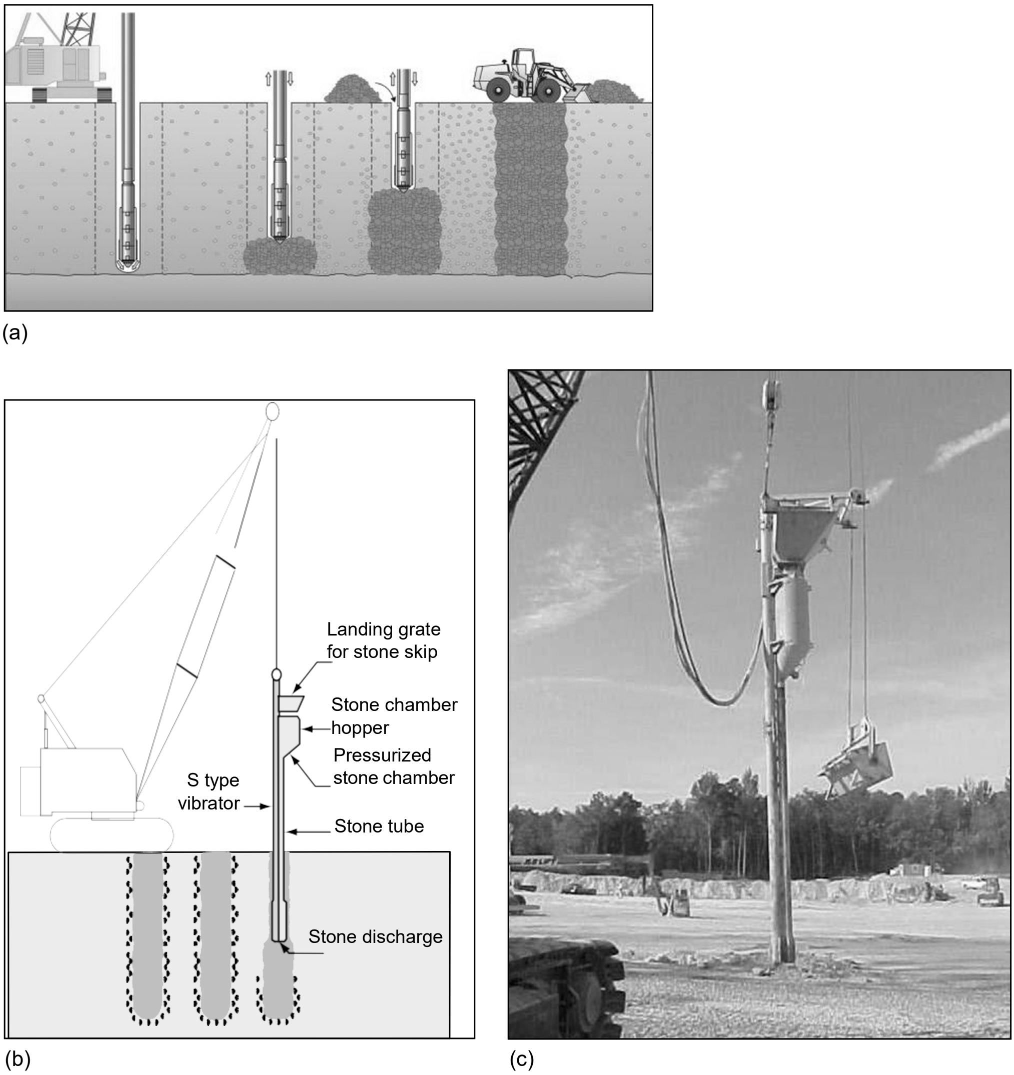


FIGURE 12.11
Stone column construction: (a) wet top feed method, (b) schematic, and (c) field implementation of dry bottom feed method. (From Hayward Baker Inc. With permission.)

Quality control and quality assurance: During production, important parameters to monitor and document include location, depth, ammeter increases (see Section 12.2.2), and quantity of stone backfill used. Post-treatment penetration testing can be performed to measure the improvement achieved in granular soils. Full-scale load tests are becoming common with test footings measuring as large as 10 ft square (3.1 m) and loaded to 150% of the design load (Figure 12.13).

12.3.2 Vibro Concrete Columns

Vibro concrete columns (VCCs) involve constructing concrete columns *in situ* using a bottom feed vibroflot (Figure 12.14). The method will densify granular soils and transfer

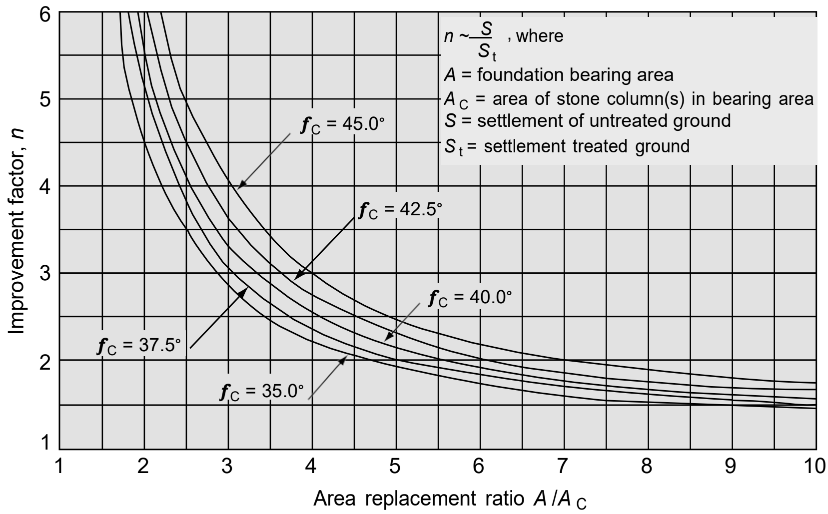


FIGURE 12.12
Chart to estimate improvement factor with stone columns.

loads through soft cohesive and organic soils. The method is used to reduce foundation settlements, to increase bearing capacity, to increase slope stability, and as an alternative to piling.

Applicable soil types: VCCs are best suited to transfer area loads, such as embankments and tanks, through soft and/or organic layers to an underlying granular layer. The depth of the groundwater table is not critical.

Equipment: The equipment is similar to the bottom feed stone column setup. A concrete hose connects a concrete pump to the top of the tremie pipe. Since verticality is important, the vibroflot is often mounted in a set of leads or a spotter.

Procedure: The vibroflot is lowered or pushed through the soft soil until it penetrates into the bearing stratum. Concrete is then pumped as the vibroflot is repeatedly raised and lowered about 2 ft (0.75 m) to create an expanded base and densifying surrounding granular soils. The concrete is pumped as the vibroflot is raised to the surface. At the

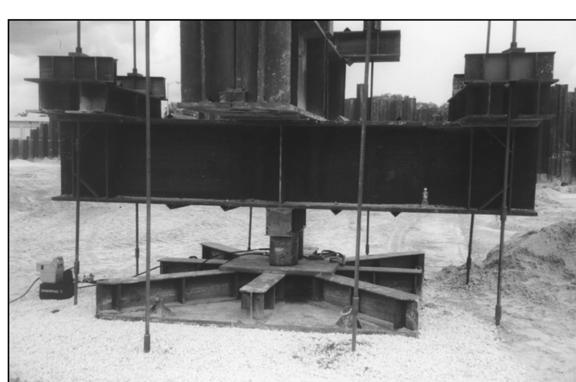
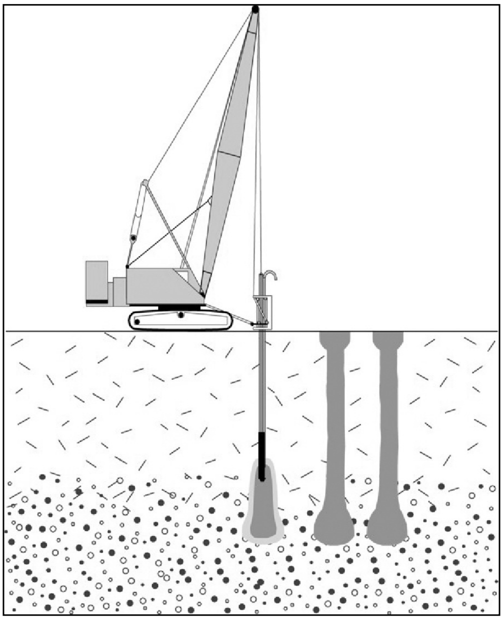


FIGURE 12.13
Full-scale load test (10 ft or 3.1 m², loaded to 15 ksf or 719 kPa). (From Hayward Baker Inc. With permission.)



(a)

(b)

FIGURE 12.14
Installation of vibro concrete columns: (a) schematic, (b) field implementation. (From Hayward Baker Inc. With permission.)

ground surface, the vibroflot is again raised and lowered several times to form an expanded top. Most VCC applications are less than 40 ft (12.3 m) in depth.

Materials: Concrete or cement mortar grout is typically used. The mix design depends on the requirements of the application.

Design: The analysis and design of VCCs are essentially the same as would be performed for an expanded base pile except that the improved soil parameters are used.

Quality control and quality assurance: During production, important parameters to monitor and document include location, depth, verticality, injection pressure and quantity, and concrete quality. It is very important to monitor the pumping and extraction rates to verify that the grout pumping rate matches or slightly exceeds the rate at which the void is created as the vibroflot is extracted. VCCs can be load tested in accordance with ASTM D 1143.

12.3.3 Soil Nailing

Soil nailing is an *in situ* technique for reinforcing, stabilizing, and retaining excavations and deep cuts through the introduction of relatively small, closely spaced inclusions (usually steel bars) into a soil mass, the face of which is then locally stabilized (Figure 12.15). The technique has been used for four decades in Europe and more recently in the United States. A zone of reinforced ground results that functions as a soil retention system. Soil nailing is used for temporary or permanent excavation support/retaining walls, stabilization of tunnel portals, stabilization of slopes, and repairing retaining walls.

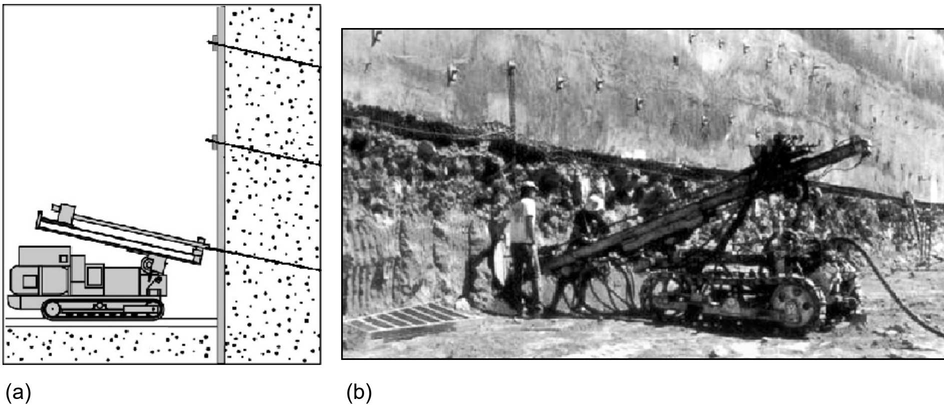


FIGURE 12.15
Soil nailing: (a) schematic, (b) field implementation. (From Hayward Baker Inc. With permission.)

Applicable soil types: The procedure requires that the soil temporarily stand in a near vertical face until a row of nails and facing are installed. Therefore, cohesive soil or weathered rock is best suited for this technique. Soil nails are not easily performed in cohesionless granular soils, soft plastic clays, or organics/peats.

Equipment: The technique requires some piece of earth moving equipment (such as a dozer or backhoe) to excavate the soil, a drill rig to install the nails, a grout mixer and pump (for grouted nails), and a shotcrete mixer and pump (if the face is to be stabilized with shotcrete).

Procedure: The procedure for constructing a soil nail excavation support wall is a top down method (Figure 12.16). A piece of earth moving equipment (such as a dozer or backhoe) excavates the soil in incremental depths, typically 3 to 6 ft (1 to 2 m). Then a drill rig typically is used to drill and grout the nails in place, typically on 3 to 6 ft (1 to 2 m) centers. After each row of nails is installed, the excavated face is stabilized, typically by fastening a welded wire mesh to the nails and then placing shotcrete.

Materials: Soil nails are typically steel reinforcing bars but may consist of steel tubing, steel angles, or high-strength fiber rods. Grouted nails are usually installed with a Portland

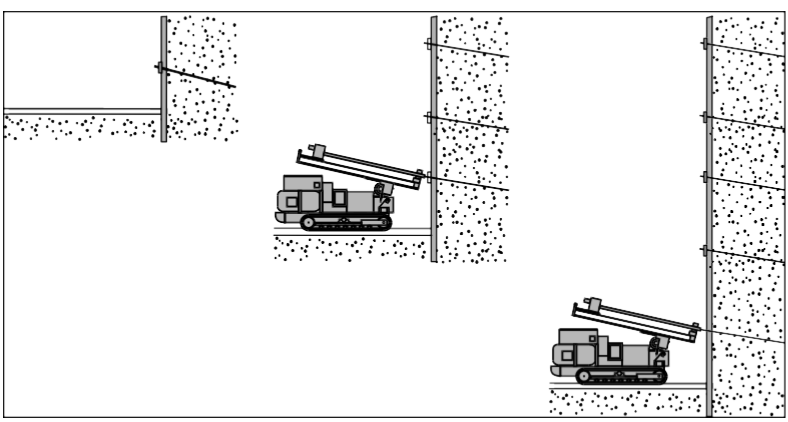


FIGURE 12.16
Soil nailing process. (From Hayward Baker Inc. With permission.)

cement grout slurry. The facing can be prefabricated concrete or steel panels, but is usually shotcrete, reinforced with welded wire mesh, rebar or steel or polyester fibers.

Design: Soil nails are designed to give a soil mass an apparent cohesion by transferring of resisting tensile forces generated in the inclusions into the ground. Frictional interaction between the ground and the steel inclusions restrain the ground movement. The main engineering concern is to ensure that the ground–inclusion interaction is effectively mobilized to restrain ground displacements and can secure the structural stability with an appropriate factor of safety. There are two main categories of design methods:

1. Limit equilibrium design methods
2. Working stress design methods.

Many software design programs are available including one developed in 1991 by CALTRANS called Snail.

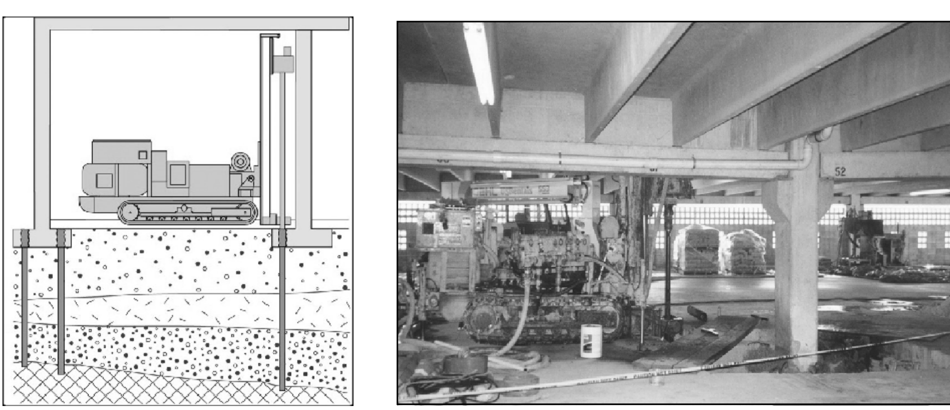
Soil nail walls are generally not designed to withstand fluid pressures. Therefore, drainage systems are incorporated into the wall, such as geotextile facing, or drilled in place relief wells and slotted plastic collection piping. Surface drainage control above and behind the retaining wall is also critical.

Extreme care should be exercised when an existing structure is adjacent to the top of a soil nail wall. The soil nail reinforced mass tends to deflect slightly as the mass stabilizes under the load. This movement may cause damage to the adjacent structure.

Quality control and quality assurance: The location and lengths of the nails are important to monitor and document. In addition, the grout used in the installation of grouted nails can be sampled and tested to confirm that it exceeds the design strength. Tension tests can also be performed on test nails to confirm that the design bond is achieved.

12.3.4 Micropiles

Micropiles, also known as minipiles and pin piles, are used in almost any type of ground to transfer structural load to competent bearing strata (Figure 12.17). Micropiles were originally small diameter (2 to 4 in., or 5 to 10 cm), low-capacity piles. However, advances



(a) (b)

FIGURE 12.17
Micropiling: (a) schematic, (b) field implementation. (From Hayward Baker Inc. With permission.)

in drilling equipment have resulted in design load capacities in excess of 300 tons (2.7 MN) and diameters in excess of 10 in. (25 cm). Micropiles are often installed in restricted access and limited headroom situations. Micropiles can be used for a wide range of applications; however, the most common applications are underpinning existing foundations or new foundations in limited headroom and tight access locations.

Applicable soil types: Since micropiles can be installed with drilling equipment and can be combined with different grouting techniques to create the bearing element, they can be used in nearly any subsurface soil or rock. Their capacity will depend on the bearing soil or rock.

Equipment: The micropile shaft is usually driven or drilled into place. Therefore, a drill rig or small pile driving hammer on a base unit is required. The pipe is filled with a cement grout so the appropriate grout mixing and pumping equipment is required. If the bearing element is to be created with compaction grout or jet grout, the appropriate grouting equipment is also required.

Procedure: The micropile shaft is usually either driven or drilled into place. Unless the desired pile capacity can be achieved in end bearing and side friction along the pipe, some type of bearing element must be created (Figure 12.18). If the tip is underlain by rock, this could consist of drilling a rock socket, filling the socket with grout and placing a full-length, high-strength threaded bar. If the lower portion of the pipe is surrounded or underlain by soil, compaction grouting or jet grouting can be performed below the bottom of the pipe. Also, the pipe can be filled with grout which is pressurized as the pipe is partially extracted to create a bond zone. The connection of the pipe to the existing or planned foundation must then be constructed.

Materials: The micropile typically consists of a steel rod or pipe. Portland cement grout is often used to create the bond zone and fill the pipe. A full length steel threaded bar is also common, composed of grade 40 to 150 ksi steel. In some instances, the micropile only consists of a reinforced, grout column.

Design: The design of the micropile is divided into three components: the connection with the existing or planned structure, the pile shaft which transfers the load to the bearing zone, and the bearing element which transfers the load to the soil or rock bearing layer.

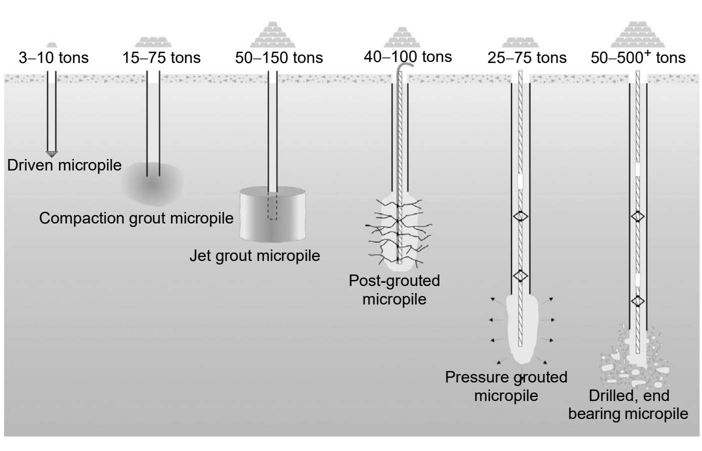


FIGURE 12.18
Sample of micropile bearing elements. (From Hayward Baker Inc. With permission.)

A standard structural analysis is used to design the pile section. If a grouted friction socket is planned, Table 12.5 can be used to estimate the sockets diameter and length. Bond lengths in excess of 30 ft (9.2 m) do not increase the piles capacity.

Quality control and quality assurance: During the construction of the micropile, the drilling penetration rate can be monitored as an indication of the stratum being drilled. Grout should be sampled for subsequent compressive strength testing. The piles verticality and length should also be monitored and documented.

A test pile is constructed at the beginning of the work and load tested to 200% of the design load in accordance with the standard specification ASTM D 1143 (Figure 12.19).

12.3.5 Fracture Grouting

Fracture grouting, also known as compensation grouting, is the use of a grout slurry to hydro-fracture and inject the soil between the foundation to be controlled and the process causing the settlement (Figure 12.20). Grout slurry is forced into soil fractures, thereby causing an expansion to take place counteracting the settlement that occurs or producing a controlled heave of the foundation. Multiple, discrete injections at multiple elevations can create a reinforced zone. The process is used to reduce or eliminate previous settlements, or to prevent the settlement of structures as underlying tunneling is performed.

A variation of fracture grouting is injection systems for expansive soils. The technique reduces the post-treatment expansive tendencies of the soil by either raising the soils' moisture content, filling the desiccation patterns in the clay or chemically treating the clay to reduce its affinity to water.

Applicable soil types: Since the soil is fractured, the technique can be performed in any soil type.

Equipment: For fracture grouting, the equipment consists of a drill rig to install the sleeve port pipes, grout injection tubing with packers, grout mixer, and a high-pressure grout

TABLE 12.5

Estimated Soil and Rock Bond Values for Micropiles

Soil/Rock Description	SPT *N* value (blows/ft)	Grout Bond with Soil/Rock (ksf)[a]
Nonpressure grouted		
Silty clay	3–6	0.5–1.0
Sandy clay	3–6	0.7–1.0
Medium clay	4–8	0.75–1.25
Firm clay or stiffer	>8	1.0–1.5
Sands	10–30	2–4
Soft shales		5–15
Slate and hard shales		15–28
Sandstones		15–35
Soft limestone		15–33
Hard limestone		20–35
Pressure grouted		
Medium dense sand		3.5–6.5
Dense sand		5.5–8.5
Very dense sand		8–12

[a]Design values, 1 ft ¼ 0.308 m, 1 ksf ¼ 47.9 kPa.

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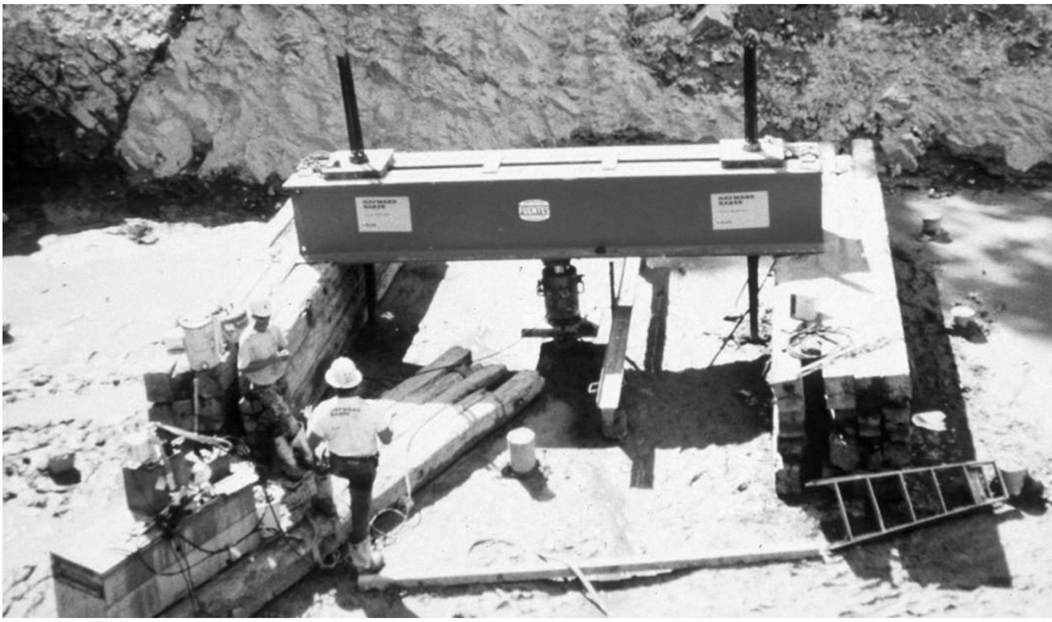
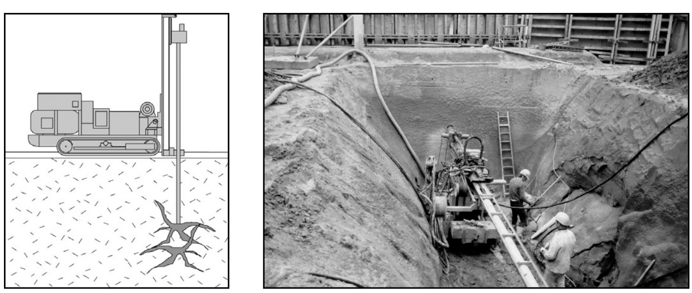


FIGURE 12.19
Micropile load test. (From Hayward Baker Inc. With permission.)



(a) (b)

FIGURE 12.20
Fracture grouting: (a) schematic, (b) field implementation. (From Hayward Baker Inc. With permission.)

pump. A sleeve port pipe is a steel or PVC pipe with openings at regular intervals along its length to permit grout injection at multiple locations along the pipes length. Also a precise real-time level surveying system is often required to measure the movements of the structure or the ground surface.

For injection of expansive soils, the equipment generally consists of a track mounted rig that pushes multiple injection pipes into the ground at the same time (Figure 12.21). A mixing plant, storage tank and pump prepare, store, and deliver the solution to be injected.

Procedure: For fracture grouting beneath existing structures, large diameter shafts (10 to 15 ft, or 3 to 4.6 m, in diameter) or pits are constructed adjacent to the exterior of the structure to be controlled. From these shafts, a drill rig installs the sleeve port pipes horizontally beneath the structure. Then a grout injection tube is inserted into the sleeve port pipe. Packers on the injection tube are inflated on either side of an individual port and grout is injected. The packers are then deflated, the injection tube moved to another port, and the process repeated as necessary to achieve either the desired heave or

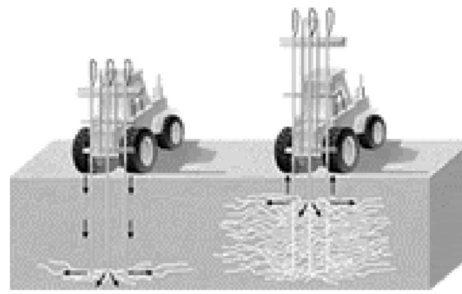


FIGURE 12.21
Injection rig for treatment of expansive soils. (From Hayward Baker Inc. With permission.)

prevent settlement. A level surveying system provides information on the response of the ground and overlying structure which is used to determine the location and quantity of the grout to be injected.

For injection of expansive soils, multiple injection rods are typically pushed into the ground to the desired treatment depth (typically 7 to 12 ft, or 2.2 to 3.7 m) and then an aqueous solution is injected as the rods are extracted.

Materials: For fracture grouting beneath structures, the grout typically consists of Portland cement and water.

For injection of expansive soils, the following solutions have been used:

Water — used to swell expansive clays as much as possible prior to construction.

Lime and fly ash — used to fill the desiccation pattern of cracks, reducing the avenues of moisture change.

Potassium chloride and ammonium lignosulfonate — used to chemically treat the clay and reduce its affinity for water.

Design: For fracture grouting beneath a structure, the design involves identifying the strata which has or will result in settlement, and placing the injection pipes between the shallowest stratum and the structure. For injection of expansive soils, the design includes identifying the lateral and vertical extents of the soils requiring treatment.

Quality control and quality assurance: For fracture grouting beneath existing structures, it is critical to know where all the injection ports are located, both horizontally and vertically. The monitoring of the overlying structure is then critical so that the affected portion of the structure is accurately identified and the injection is performed in the correct ports.

For injection of expansive soil, acceptance is typically based on increasing the *in situ* moisture content to the plastic limit plus 2 to 3 moisture points, reducing pocket penetrometer readings to 3 tsf (288 kPa) or less, and reducing the average swell to 1% or less within the treatment zone.

12.3.6 Infrequently-Used Reinforcement Techniques

12.3.6.1 Fibers and Biotechnical

Fiber reinforcement consists of mixing discrete, randomly oriented fibers in soil to assist the soil in tension. The use of fibers in soil dates back to ancient time but renewed interest was generated in the 1960s. Laboratory testing and computer modeling have been performed; however, field testing and evaluation lag behind. There are currently no standard guidelines on field mixing, placement and compaction of fiber-reinforced soil composites.

Biotechnical reinforcement involves the use of live vegetation to strengthen soils. This technique is typically used to stabilize slopes against erosion and shallow mass movements. The practice has been widely used in the United States since the 1930s. Recent applications have combined inert construction materials with living vegetation for slope protection and erosion control. Research has been sponsored by the National Science Foundation to advance the practice.

12.4 Fixation

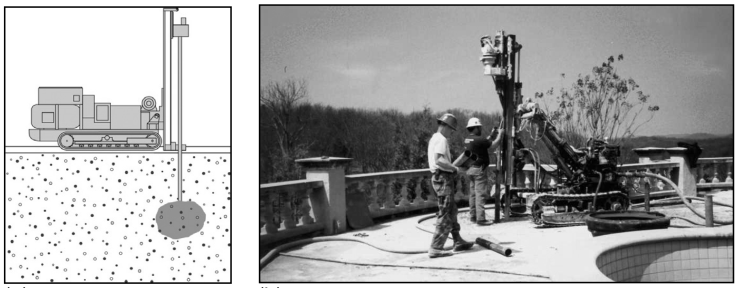
12.4.1 Permeation Grouting

Permeation grouting is the injection of a grout into a highly permeable, granular soil to saturate and cement the particles together. The process is generally used to create a structural, load carrying mass, a stabilized soil zone for tunneling, and water cutoff barrier (Figure 12.22).

Applicable soil types: The permeability requirement restricts the applicable soils to sands and gravels, with less than 18% silt and 2% clay. The depth of the groundwater table is not critical in free-draining soils, since the water will be displaced as the grout is injected. Loose sands will have reduced strengths when grouted compared to sands with SPT *N* values of 10 or greater.

Equipment: The mixing plant and grout pump vary depending on the type of grout used. Drill rigs typically install the grout injection pipe. The rigs can vary from very small to very large, depending on the project requirements. When the geometry of the grouted mass is critical, sleeve port pipes will be used.

Procedure: The grout can be mixed in batches (cementacious slurries) or stream mixed (silicates and other chemical grouts). Batch mixing involves mixing a selected volume of grout, possibly 1 yard^3 or 0.79 m^3, and then injecting it before the next batch is mixed. The amount batched depends on the speed of injection and amount of time the specific grout



(a) (b)

FIGURE 12.22
Permeation grouting: (a) schematic, (b) field implementation. (From Hayward Baker Inc. With permission.)

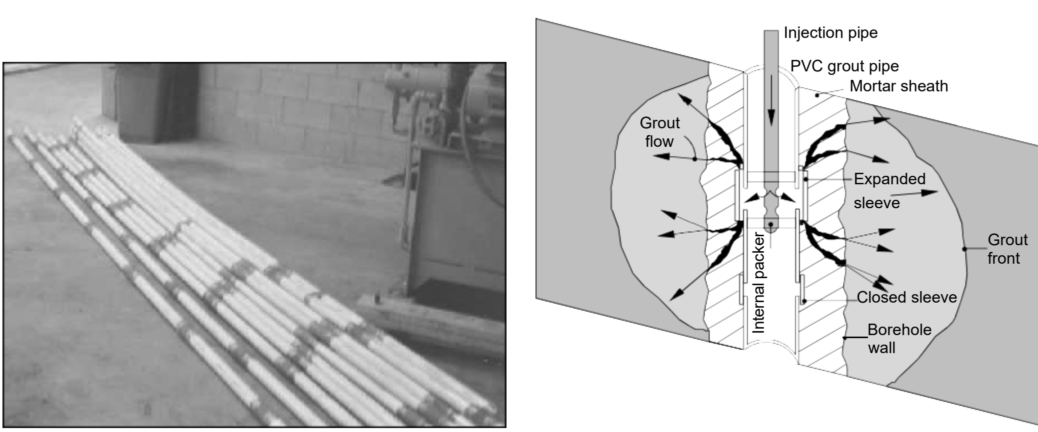
can be held and still be usable. Steam mixing involves storing the grout components in several tanks and then pumping them through separate hoses that combine before the grout reaches the injection pipe. If the geometry of the grouted mass is not important, the grout can be pumped through and out the bottom of the injection pipe. The pipe is then raised in increments, 1 to 3 ft (0.3 to 0.9 m), as the specified volume is injected at each interval.

A sleeve port pipe is used when the grouted geometry is important, such as excavation support walls. A sleeve port pipe is a steel or PVC pipe with holes, or ports, located at regular intervals, possibly 1 to 3 ft (0.3 to 0.9 m), along its length. A thin rubber membrane is placed over each port. The rig drills a hole in the soil, fills it with a weak, brittle, Portland cement grout, and inserts the sleeve port pipe. After the weak grout has hardened, a grout injection pipe with two packers is inserted into the sleeve port pipe allowing the grout to be injected through one port at a time (Figure 12.23). The injection pipe is then raised or lowered to another port and the process repeated in a sequence that includes primary, secondary, and tertiary injections.

Materials: The type of grout used depends on the application and soil grain size. For structural applications in gravel, Portland cement and water can be used. However, the particle size of the Portland cement is too large for sands. A finely ground Portland cement is available for use in course to medium sands. In fine, medium, and coarse sand, chemical grout can be used. The most common chemical grout used for structural applications is sodium silicate. Other chemical grouts are acrylates and polyurethanes.

Design: Generally, unconfined compressive strength and permeability are the design parameters. Sands grouted with sodium silicate can achieve a permeability of 1 10^{-5} cm/sec and an unconfined compressive strength of 50 to 300 psi (0.345 kPa to 2.07 MPa), although consistently achieving values in the field greater than 100 psi (0.69 MPa) is difficult. A standard analysis is performed assuming that the grouted soil is a mass with the design parameters. For excavation support walls, the mass is analyzed as a gravity structure, calculating the shear, sliding and overturning of the mass, as well as the global stability of the system.

Quality control and quality assurance: The mix design of the grouted soil can be estimated in the lab by compacting the soil to be grouted in a cylinder or cube molds at about the same



(a) (b)

FIGURE 12.23
(a) Sleeve port pipes and (b) cross section of grout injection through a port. (From Hayward Baker Inc. With permission.)

density as exists *in situ* and then saturating the soil with the grout. Laboratory permeability or unconfined compressive strength tests can be performed after a specified cure time, such as 3, 7, 14, and 28 days. During production, the grout volume and pressure should be monitored and documented. The grouted soil can also be cored and tested after grouting.

12.4.2 Jet Grouting

Jet grouting (Figure 12.24) was conceived in the mid-1970s and introduced in the United States in the 1980s. The technique hydraulically mixes soil with grout to create *in situ* geometries of soilcrete. Jet grouting offers an alternative to conventional grouting, chemical grouting, slurry trenching, underpinning, or the use of compressed air or freezing in tunneling. A common application is underpinning and excavation support of an existing structure prior to performing an adjacent excavation for a new, deeper structure.

Super jet grouting is a modification to the system allowing creation of large diameters (11 to 16 ft, or 3.4 to 4.9 m) and is efficient in creating excavation bottom seals and treatment of specific soil strata at depth.

Applicable soil types: Jet grouting is effective across the widest range of soils. Because it is an erosion-based system, soil erodibility plays a major role in predicting geometry, quality, and production. Granular soils are the most erodible and plastic clays the least. Since the soil is a component of the final mix, the soil also affects the soilcrete strength (Figure 12.25). Organic soils are problematic and can be the cause for low strengths unless partially removed by an initial erosion pass before grouting. Flowing water can also be a problem.

Equipment: An on-site batch plant is required to mix the grout as needed. Pumps are also required to pump the grout and sometimes water and air to the drill rig. The drill rig is necessary to flush the jet grout monitor into the ground. Compact drills are capable of low headroom and tight access work. Pumps may also be required to remove the soilcrete waste.

Procedure: Jet grout is a bottom-up process (Figure 12.26). The drill flushes the monitor to the bottom of the treatment zone. The erosion and grout jets are then initiated as the monitor is rotated and extracted to form the soilcrete column. Varying geometries can be formed. Rotating the monitor through only a portion of a circle will create a portion of a column. Extracting the monitor without rotating it will create a panel. Treatment depths greater than 60 ft (18.5 m) require special precautions.

(a) (b)

FIGURE 12.24
Jet grouting: (a) schematic, (b) field implementation. (From Hayward Baker Inc. With permission.)

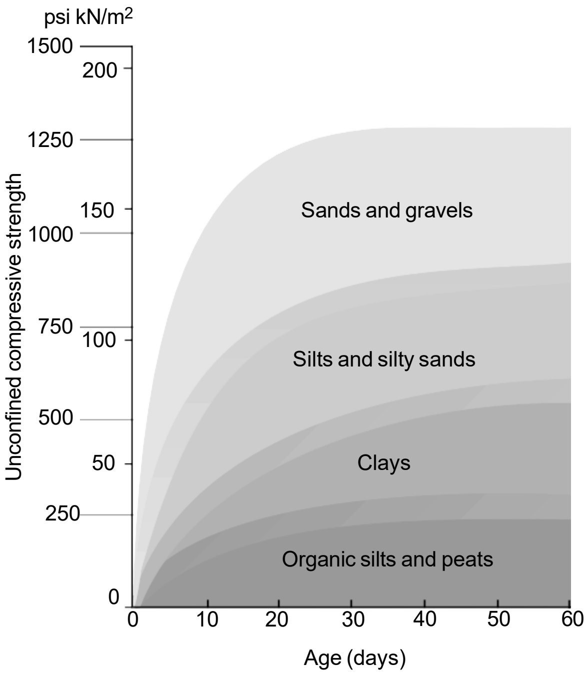


FIGURE 12.25
Range of soilcrete strengths based on soil type. (From Hayward Baker Inc. With permission.)

There are three traditional jet grout systems (Figure 12.27). Selection of the most appropriate system is determined by the *in situ* soil, the application, and the required strength of the soilcrete. The three systems are single, double, and triple fluid.

The single-fluid system uses only a high-velocity cement slurry grout to erode and mix the soil. This system is most effective in cohesionless soil and is generally not an appropriate underpinning technique because of the risk of pressurizing and heaving the ground.

The double-fluid system surrounds the high-velocity cement slurry jet with an air jet. The shroud of air increases the erosion efficiency. Soilcrete columns with diameters over 3 ft (0.9 m) can be achieved in medium to dense soils, and more than 6 ft (1.8 m) in loose soils. The double-fluid system is more effective in cohesive soils than the single-fluid system.

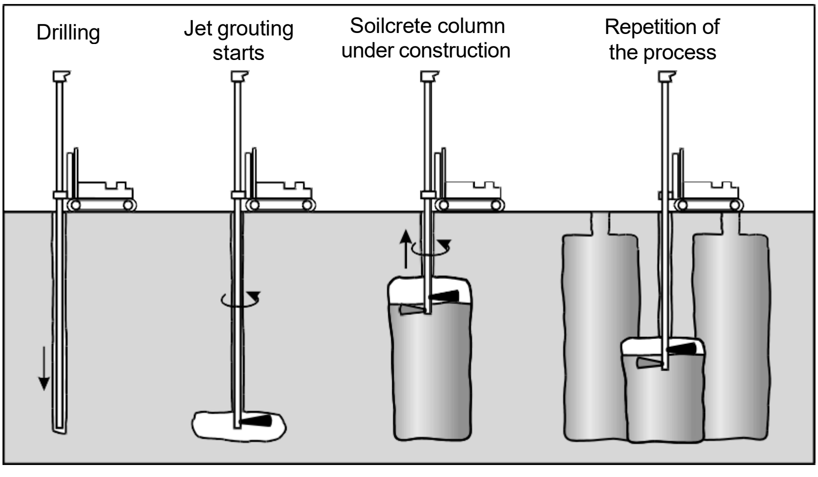


FIGURE 12.26
Jet grout process. (From Hayward Baker Inc. With permission.)

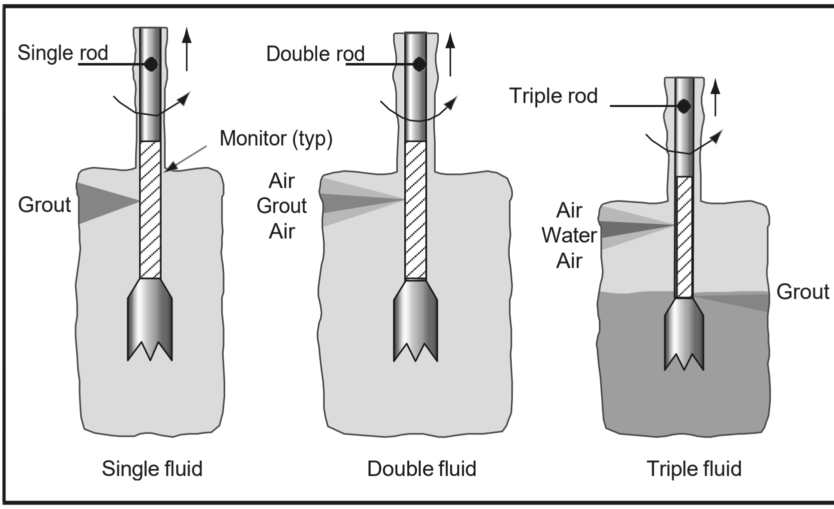


FIGURE 12.27
Single-, double-, and triple-jet grout systems. (From Hayward Baker Inc. With permission.)

The triple-fluid system uses a high-velocity water jet surrounded by an air jet to erode the soil. A lower jet injects the cement slurry at a reduced pressure. Separating the erosion process from the grouting process results in higher quality soilcrete and is the most effective system in cohesive soils.

Since material is pumped into the ground and mixed with the soil, the final mixed product has a larger volume than the original *in situ* soil. Therefore, as the mixing is performed, the excess soilcrete exits to the ground surface through the annulus around the drill steel. This waste material must be pumped or directed to an onsite retention area or trucked off-site. Since the waste contains cement, the waste sets up overnight and can be handled as a solid the following day.

Materials: Portland cement and water are generally the only two components, although additives can be utilized.

Design: Generally, either unconfined compressive strength or permeability is the design parameter. A standard analysis is performed to determine the required soilcrete geometry necessary based on the parameters achievable in the soil to be mixed. For excavation support walls, the mass must resist the surcharge, soil and water pressure imposed after excavation. This may include analysis of shear, sliding and overturning, as well as the global stability of the system. For underpinning applications, a standard bearing capacity and settlement analysis is performed as would be done for any cast in place pier.

Quality control and quality assurance: Monitoring and documenting the production parameters and procedures is important to assure consistency and quality. Test cylinders or cubes made from the waste material give a conservative assessment of the *in situ* characteristics. Wet sampling of the soilcrete *in situ* can also be performed although it is problematic. Coring of the hardened soilcrete is typical.

12.4.3 Soil Mixing

Soil mixing mechanically mixes soil with a binder to create *in situ* geometries of cemented soil. Mixing with a cement slurry was originally developed for environmental applications; however, advancements have reduced the costs to where the process is used for many general civil works, such as *in situ* walls, excavation support, port development on soft sites, tunneling support, and foundation support. Mixing with dry lime and cement was developed in the Scandinavian countries to treat very wet and soft marine clays.

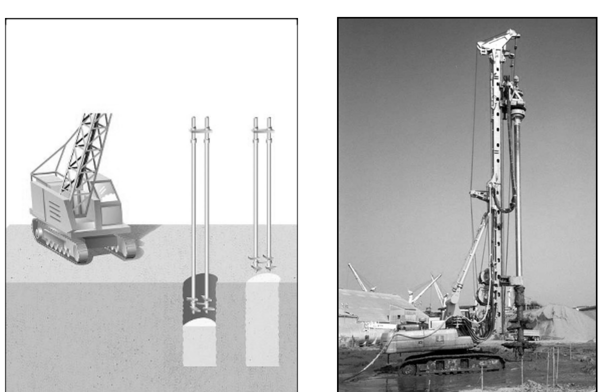
Applicable soil types: The system is most applicable in soft soils. Boulders and other obstructions can be a problem. Cohesionless soils are easier to mix than cohesive soils. The ease of mixing cohesive soils varies inversely with plasticity and proportionally with moisture content. The system is most commonly used in soft cohesive soils as other soils can often be treated more economically with other technologies. Organic soils are problematic and generally require much larger cement content. The quality achieved with soil mixing is slightly lesser than that achieved with jet grouting in the same soils, with unconfined compressive strengths between 10 and 500 psi (0.69 to 3.45 MPa), and permeabilities as low as 1 10^{-7} cm/sec, depending on the soil type and binder content.

Equipment: A high-volume batching system is required to maintain productivity and economics. The components consist of an accurately controlled mixer, temporary storage, and high-volume pumps.

A drilling system is required to turn the mixing tool in the ground. The system varies from conventional hydraulic drill heads to dual-motor, crane-mounted turntables with torque requirements ranging from 30,000 to 300,000 ft lb (41 to 411 kJ). Multiaxis, electrically powered drill heads are also used, primarily for walling applications.

The mixing tool is generally a combination of partial flighting, mix blades, injection ports and nozzles, and shear blades. It can be a single- or multiple-axis tool (Figure 12.28). Tool designs vary with soil types and are often custom-built for specific projects (Figure 12.29). The diameter of the tool can vary from 1.5 to 12 ft (0.46 to 3.7 m).

Procedure: The binder is injected as the tool is advanced down to assist in penetration and to take advantage of this initial mixing. The soil and binder are mixed a second time as the tool is extracted. The rate of penetration and extraction is controlled to achieve adequate mixing. Single columns or integrated walls are created as the augers are worked in overlapping configurations. Treatment depths as great as 100 ft (31 m) have been achieved.



(a) (b)

FIGURE 12.28
Soil Mixing: (a) Schematic, (b) Field implementation.

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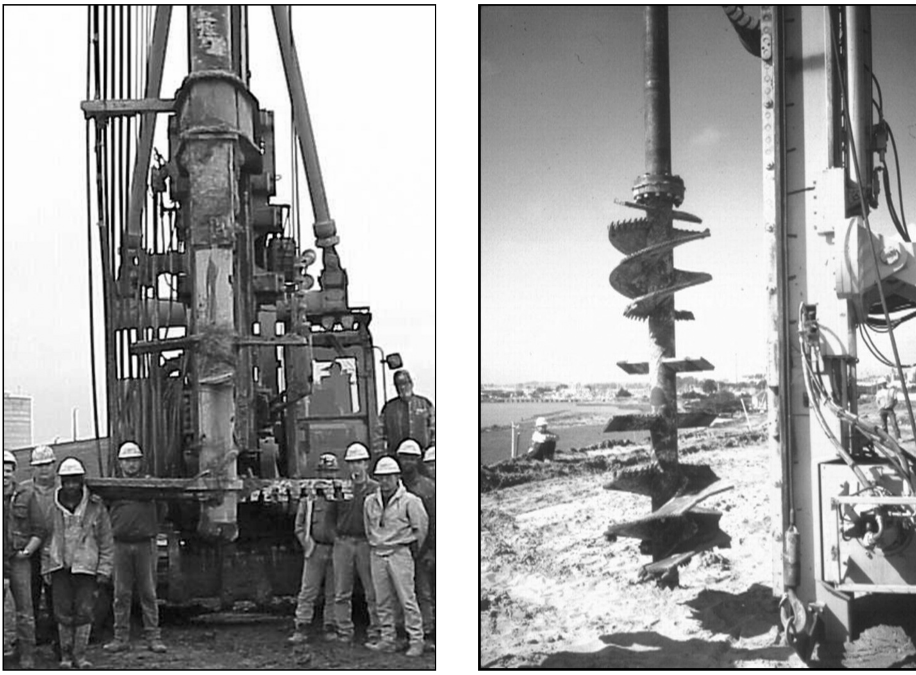


FIGURE 12.29
Example of soil mixing tools. (From Hayward Baker Inc. With permission.)

Materials: For wet soil mixing, the binder is delivered in a slurry form. Slurry volumes range from 20 to 40% of the soul volume being mixed. Common binders are Portland cement, fly ash, ground blast furnace slag, and additives. For dry soil mixing, the same materials (also line) are pumped dry using compressed air. Preproduction laboratory testing is used to determine mix energy and grout proportions.

Design: As with jet grouting, unconfined compressive strength and permeability are generally the design parameters. A standard analysis is performed to determine the required geometry based on the parameters achievable in the soil to be mixed. For excavation support walls, the mass can be designed as a standard excavation wall, or a thicker mass can be created and analyzed as a gravity structure, calculating the mass' shear, sliding and overturning, as well as the global stability of the system. When used as structural load bearing columns, a standard bearing capacity and settlement analysis is performed as would be for any cast in place pier. Anchored retention using steel reinforcement is common for support walls.

12.4.3.1 Dry Soil Mixing

Dry soil mixing (Figure 12.30) is a low-vibration, quiet, clean form of ground treatment technique that is often used in very soft and wet soil conditions and has the advantage of producing very little spoil. The high speed rotating mixing tool is advanced to the maximum depth, ''disturbing'' the soil on the way down. The dry binder is then pumped with air through the hollow stem as the tool is rotated on extraction. It is very effective in soft clays and peats. Soils with moisture content, greater than 60% are most economically

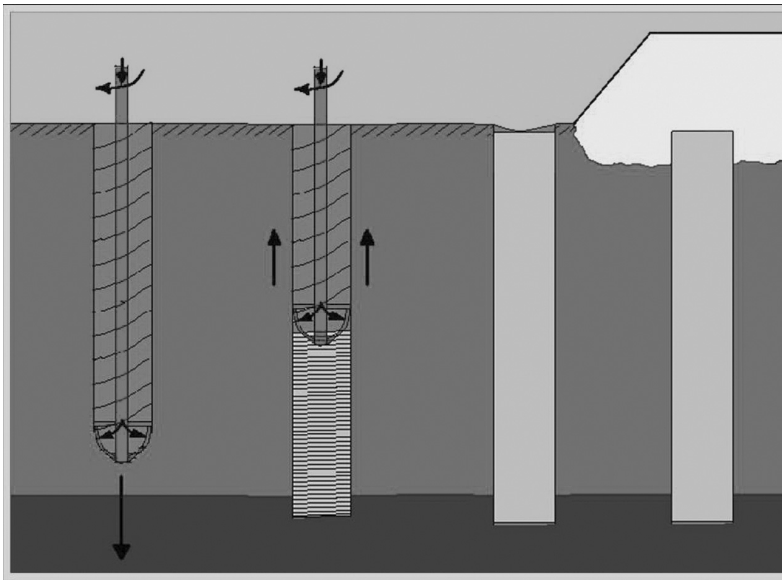


FIGURE 12.30
Illustration of dry soil mixing technique. (From Hayward Baker Inc. With permission.)

treated. This process uses cementacious binders to create bond among soil particles and thus increases the shear strength and reduces the compressibility of weak soils.

The most commonly used binding agents are cement, lime, gypsum, or slag. Generally, the improvement in shear strength and compressibility increases with the binder dosage. By using innovative mixtures of different binders engineers usually achieve improved results. It is known that strength gains are optimum for inorganic soils. It is realized that the strength gain would decrease with increasing organic and water content. The binder content varies from about 5 lb/ft^3 for soft inorganic clays to about 18 lb/ft^3 for peats with a high organic content.

12.4.3.2 Wet Soil Mixing

Wet soil mixing (Figure 12.31) is a similar technique except that a slurry binder is used making it more applicable with dryer soils (moisture contents less than 60%). The grout slurry is pumped through the hollow stem to the trailing edge of the mixing blades both during penetration and extraction. Depending on the *in situ* soils, the volume of grout slurry necessary varies from 20 to 40% of the soil volume. The technique produces a similar amount of spoil (20 to 40%) which is essentially excess mixed soil which, after setting up, can often be used as structural fill. The grout slurry can be composed of Portland cement, fly ash, and ground granulated blast furnace slag.

Quality control and quality assurance: Preproduction laboratory testing is often performed to prescribe the mixing energy and binder components and proportions. During production, it is necessary to monitor and document parameters such as mixing depth, mixing time, grout mix details, grout injection rates, volumes and pressures, tool rotation, penetration, and withdrawal rates.

Test cylinders or cubes can be cast from wet samples, but are problematic. The hardened columns can also be cored. In weaker mixes, penetration tests can be performed.

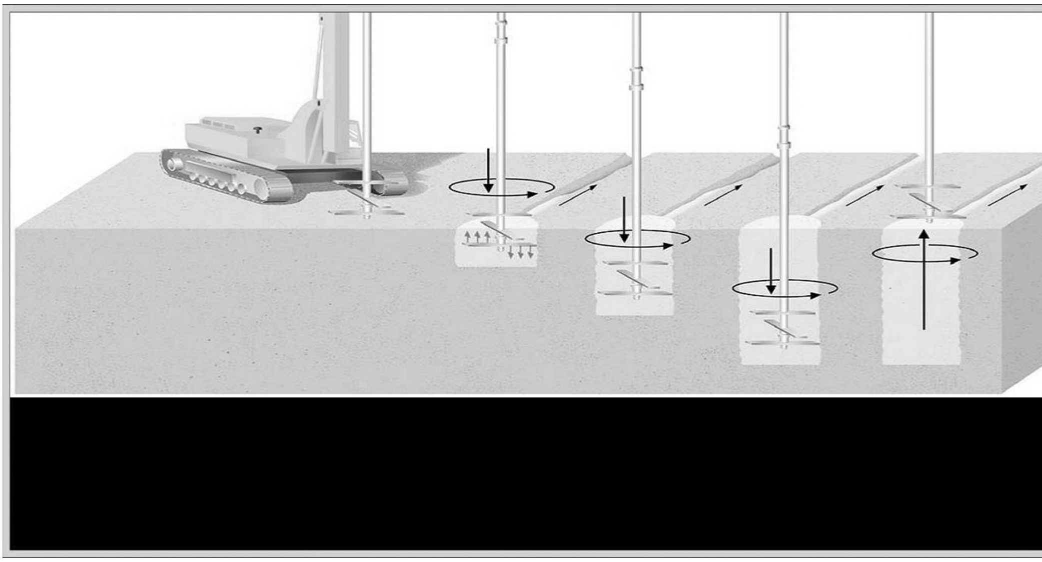


FIGURE 12.31
Illustration of wet soil mixing technique.

12.4.4 Infrequently-Used Fixation Techniques

12.4.4.1 Freezing and Vitrification

Ground freezing involves lowering the temperature of the ground until the moisture in the pore spaces freezes. The frozen moisture acts to ''cement'' the soil particles together. The first use of this technique was in 1862 in South Wales. The process typically involves placing double walled pipes in the zone to be frozen. A closed circuit is formed through which a coolant is circulated. A refrigeration plant is used to maintain the coolant's temperature. Since ice is very strong in compression, the technique has been most commonly used to create cylindrical retaining structures around planned circular excavations.

Vitrification is a process of passing electricity through graphite electrodes to melt soils *in situ*. Electrical plasma arcs have also been used and are capable of creating temperatures in excess of 40008C. The soil becomes magma, and after several days of cooling it hardens into an artificial igneous rock. Although laboratory testing is ongoing, the electrical usage of the process to date appears to make it uneconomical. It is possible that the process could find application in the field of environmental cleanup.

12.5 Other Innovative Soft-Ground Improvements Techniques

12.5.1 Rammed Aggregate Piers

Rammed aggregate piers (RAPs) are a type of stone column as presented in Section 12.3.1. Aggregate columns installed by compacting successive lifts of aggregate material in a preaugered hold (Figure 12.32). The predrilled holes, which typically have diameters of 24 to 36 in. (0.6 to 1.2 m), can extend up to about 20 ft. As seen in Figure 12.33, aggregate is compacted in lifts with a beveled tamper to create passive soil pressure conditions both at the bottom and the sides of the piers. RAPs are generally restricted to cohesive soils in

FIGURE 12.32
Installation of rammed aggregate piers, a type of stone column. (From Geopiers Foundation Co. With permission.).

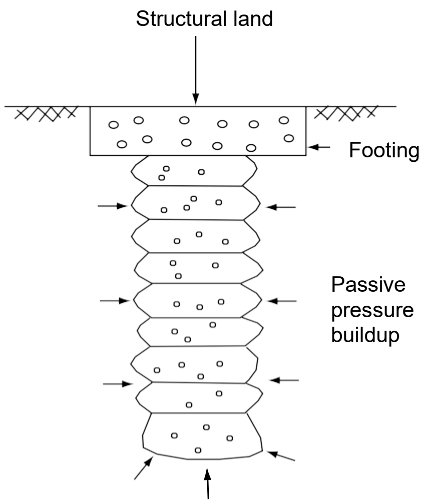


FIGURE 12.33
Schematic diagram of a rammed aggregate pier.

which a predrill hole will stay open. Although constructed differently than store columns or vibro piers (Section 12.3.1) all provide similar improvement to cohesive soils. The vertical tamping used to construct RAPs results in minimal densification in adjacent granular soils compared to vibratory probe construction.

RAPs can be used in some of the following stone column applications that are outlined below:

1. Support shallow footings in soft ground.
2. Reinforces soils to reduce earthquake-induced settlements, however, does not densify sands against liquefaction.
3. Increase drainage and consequently expedite long-term settlement in saturated fine-grained soils.
4. Increase global stability and bearing capacity of retaining walls in soft ground.
5. Improve stability of slopes if RAPs can be installed to intersect potential shear failure planes.
6. Reduce the need for steel reinforcements when RAPs are installed below concrete mat or raft foundations.

12.5.2 Reinforced Soil Foundations

Bearing capacity of foundation soils can be improved using geogrids and geosythetics placed as a continuous single layer, closely spaced continuous mutilayer set or mattress consisting of three-dimensional interconnected cells. Although standards on design of footings on reinforced soils are currently unavailable, Koerner (1998) provides some numerical guidelines on the extent of the improvement of bearing capacity and reduction of settlement. Figure 12.34(a) and (b) shows the results of laboratory tests where geotextiles were used to improve the bearing capacity of loose sands and saturated clay, respectively.

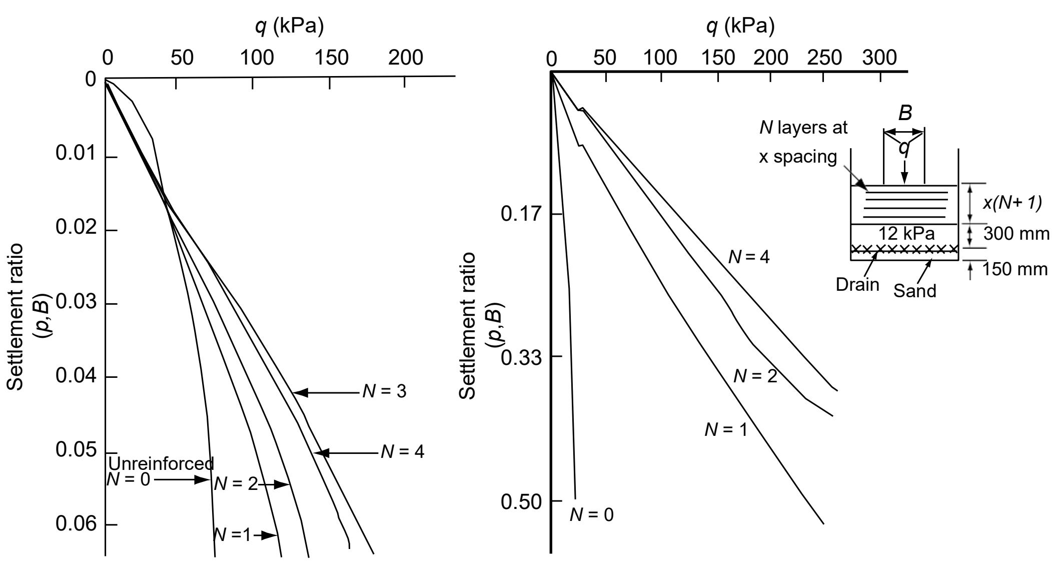


FIGURE 12.34
Improvement of soil bearing capacity with geotextiles: (a) loose sand, (b) saturated clay. (From Koerner, R., 1994. *Designing with Geosynthetics*, Prentice Hall, Englewood Cliffs, NJ. With permission.)

Figure 12.35 also shows the general approximations that the author has drawn from the results of large laboratory tests (Milligan and Love, 1984), which shows the improvement of settlement properties of saturated clay reinforced with geogrids.

A large number of load tests have been conducted in the test pits at the Turner-Fairbank Highway Research Center (TFHRC) in Alaska, USA, to evaluate the effects of single and multiple layer of reinforcement placed below shallow spread footings (FHWA, 2001). In this test program, two different geosynthetics were evaluated; a stiff biaxial geogrid and a geocell. Parameters of the testing program include: number of reinforcement layers; spacing between reinforcement layers; depth to the first reinforcement layer; plan area of the reinforcement; type of reinforcement; and soil density. Test results indicated that the use of geosynthetic reinforced soil foundations may increase the ultimate bearing capacity of shallow spread footings by a factor of 2.5 (FHWA, 2001).

12.5.2.1 Mechanisms of Bearing Capacity Failure in Reinforced Soils

In spite of the known favorable influence of geotextiles and geogrids on soil bearing capacity, the foundation designer needs to be aware of a number of mechanisms of bearing capacity failure even with reinforcements. These are discussed in Koerner (1998) as seen in Figure 12.36(a)–(d). Figure 12.36(a) shows the lack of reinforcement in the foundation influence zone while Figure 12.36(b) illustrates insufficient embedment of geotextiles or geogrids. Bearing capacity failures leading to inadequate tensile strength and excessive creep (long-term deformation) of reinforcements is shown in Figure 12.36(c) and (d), respectively.

These are discussed in Koerner (1998) as situations arising from;

- the lack of reinforcement in the foundation influence zone while Fig. 12.37
- insufficient embedment of geotextiles or geogrids.
- bearing capacity failures leading to inadequate tensile strength, and
- excessive creep (long-term deformation) of reinforcements

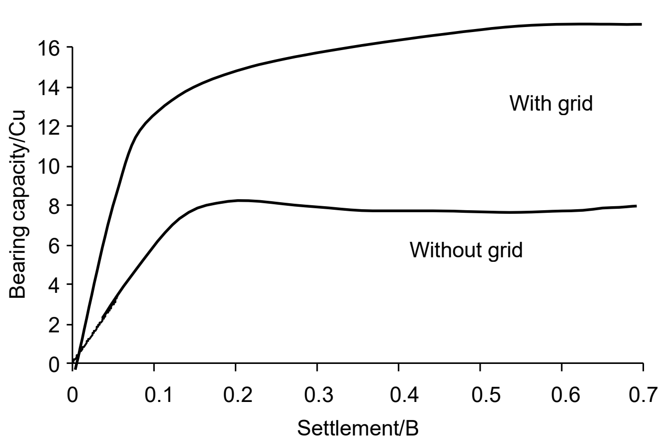
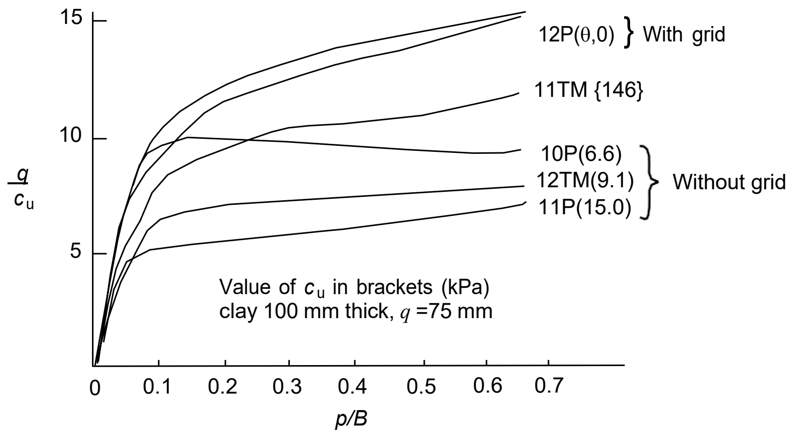


FIGURE 12.35
Improvement of settlement properties of saturated clays.



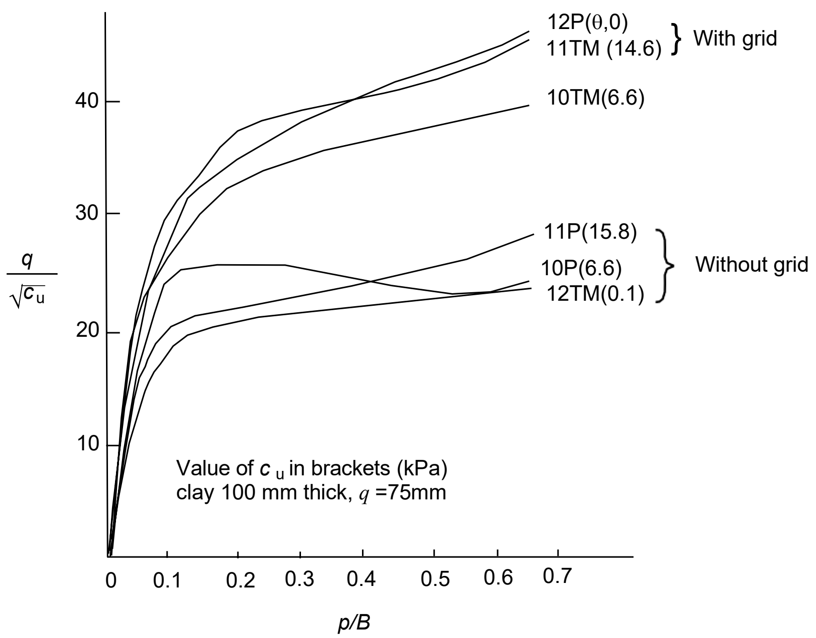


FIGURE 12.36
Improvement of settlement properties in saturated clay with geogrids. (From Koerner, R., 1994. *Designing with Geosynthetics*, Prentice Hall, Englewood Cliffs, NJ. With permission.)

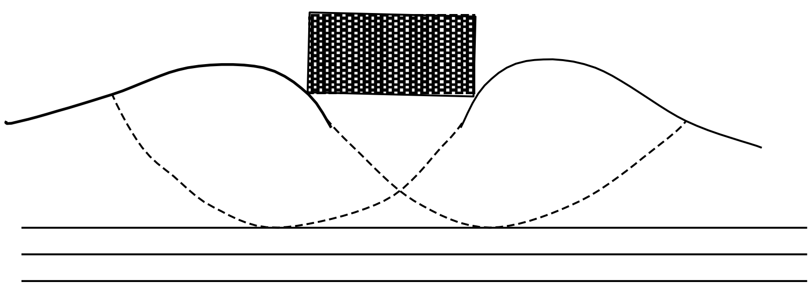


FIGURE 12.37
Lack of reinforcement in the foundation influence zone.

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