Chapter

23 Ground Improvement Techniques

LEARNING OBJECTIVES

After reading this chapter, you will be able to:

- Identify the need and objectives of ground improvement methods.
- Enlist available ground improvement methods for different types of soils and ground conditions.
- Discuss the principles of ground improvement in cohesive and cohesionless soils.
- Explain in detail how the heavy tamping technique can be used to improve the ground.
- Explain the purpose of "preloading" and "vertical drains" with neat sketches.
- Describe in detail grouting with "soil–cement mixes," "cement," and "lime" grouts.
- Explain the basic mechanism and applications of reinforced earth.
- List various materials to be used in soil reinforcement and the internal stability aspects of reinforced earth walls.
- Discuss with neat sketches the components of reinforced earth walls and list the various advantages of reinforced earth structures.
- List the types and functions of geosynthetics.
- List the various applications of geotextiles in civil engineering works.
- List various dewatering systems and explain their suitability for different soils.
- Discuss the treatment techniques for improvement of reclaimed soils containing soft clay for deeper depths.

23.1 Introduction

When the soil at the construction site is either highly compressible or weak and is unable to support a shallow foundation, deep foundations or raft foundations may be used for the structure. When a project encounters difficult foundation conditions, possible alternate solutions are as follows:

1. Avoid the particular site.
2. Design the planned structure accordingly.
3. Remove and replace unsuitable soils.
4. Attempt to modify the existing ground.

Of the above, an economical option, in many cases, will be to improve the soil at the site using suitable ground improvement techniques.

Ground improvement is generally recommended if the net loading intensity of the foundation exceeds the allowable pressure after carrying out the foundation design. Ground treatment is also needed if the resultant settlement exceeds the acceptable limits for the structure, both from the view point of distortions induced in the structure and from the operation angle, even for relative low loading intensities.

Normally, a shallow foundation is the most economical option to support a structure. If the soil at shallow depth is weak, highly compressible, or erratic, providing a shallow foundation may not be feasible, due to either a low bearing capacity or excessive total or differential settlements or an abnormally large footing size. In such a case, improving the foundation soil and providing a shallow foundation should be investigated for feasibility along with other raft or deep foundation options.

Loose cohesionless deposits in seismic zones may be prone to liquefaction during earthquakes especially under high water table conditions. In such cases, an analysis should be carried out for establishing the liquefaction potential of the subsoil. Ground improvement is used if such analysis establishes that the subsoil is prone to liquefaction.

23.2 Objectives of Ground Improvement

The following are the benefits or objectives of ground improvement techniques:

1. Increase strength.
2. Reduce distortion under stress (increases stress–strain modulus).
3. Reduce compressibility.
4. Prevent detrimental physical or chemical changes due to environmental conditions (freezing/thawing, wetting/drying).
5. Reduce susceptibility to liquefaction.
6. Reduce natural variability of borrow materials and foundation soils.
7. Enable waste containment and hazardous waste management.
8. Enable constructive use of waste materials.

23.3 Classification of Ground Improvement Methods

There are a wide variety of classifications of ground improvement methods based on different criteria. The following is the classification of ground improvement methods as per IS:13094–1992:

1. **Deep compaction of cohesionless soils:**
   - Blasting.
   - Vibro-compaction.
   - Sand compaction piles.
   - Dynamic compaction.
2. **Injection and grouting:**
   - Particulate grouting.
   - Chemical grouting.
   - Pressure-injected lime.
   - Displacement grout.
   - Electro-kinetic injection.
   - Jet grouting.
3. **Admixtures:**
   - Remove and replace.
   - Structural fills.
   - *In-situ* soil mixing.
4. **Pre-compression:**
   - Preloading with or without drains.
   - Surcharge fills.
   - Electro-osmosis.
5. **Reinforcement:**
   - Vibro-replacement.
   - Stone and sand columns.
   - Root piles and soil nailing.
   - Strips and Membranes.
6. **Thermal:**
   - Heating.
   - Freezing.

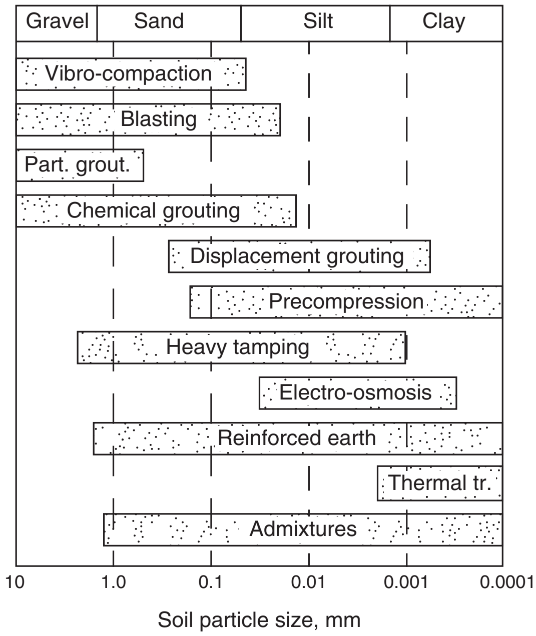

Figure 23.1 Relative suitability of some ground improvement methods.

In this chapter, ground improvement methods have been discussed under the following heads:

1. Mechanical stabilization.
2. Sand compaction piles.
3. Blasting.
4. Dynamic compaction.
5. Preloading.
6. Sand drains.
7. Prefabricated vertical drains.
8. Stone columns.
9. Reinforced earth.
10. Soil nailing.
11. Geosynthetics.
12. Foundation grouting.
13. *In-situ* soil mixing.
14. Seepage control and dewatering.
15. Ground freezing.
16. Heating.

Figure 23.1 shows the relative suitability of some of the ground improvement techniques for various soil types.

23.4 Mechanical Stabilization

Mechanical stabilization involves physical mixing of soil constituents and compaction with or without additives.

23.4.1 Principle

The objective of mechanical stabilization is to achieve maximum possible dry density of soil by suitable proportioning of materials and effective compaction, leading to high shear strength, low compressibility, and low permeability.

The desirable mechanical properties are as follows:

1. High shear strength.
2. Low compression.
3. Stability with change in moisture content.
4. Good drainage.
5. Less susceptibility to frost action.
6. Ease of compaction.

Two basic principles of mechanical stabilization are as follows:

1. Proportioning.
2. Compaction.

In this chapter, the principles of proportioning of materials are discussed. The principles, methods, equipment, and quality control for effective compaction have been already discussed in Chapter 12.

23.4.2 Mix Design in Mechanical Stabilization

The objective of mix design is to utilize available gradations to achieve maximum possible dry density of the compacted soil

23.4.2.1 Desirable Gradation

The particle size distribution that gives the maximum density may be obtained from

$$P = 100\left(\frac{d}{D}\right)^{n} \qquad (23.1)$$

where P is the cumulative % finer than size "d," D the maximum particle size, and n the gradation index = 0.3–0.5.

Fuller obtained an n value of 0.5 for spherical particles. For angular and flaky particles, the n value can vary from 0.3 to 0.5, depending on the shape factor of aggregates.

23.4.2.2 Proportioning

When different types of material are available for construction, it is necessary to mix these materials in such a proportion that would produce a mix with the highest density; for example, if three borrow areas, each with coarse aggregate, sand, and fines are available, it is first necessary to decide the best proportion of these materials. This process is called proportioning. For highway applications, the following two methods of proportioning are available:

1. Triangular chart method.
2. Rothfutch method.

Triangular Chart Method

In triangular chart method, the proportions of coarse aggregates, sand, and fines in each borrow soil are plotted on the three sides of the chart, as shown in Fig. 23.2. For example, consider three borrow areas with proportions of materials as shown in Table 23.1.

The procedure to determine the proportion of soils A, B, and C in the triangular chart method is as follows:

1. Soil A with gravel, sand, and fines content of 85%, 10%, and 5%, respectively, is plotted as point A on the triangular chart using the directions shown by the key, as shown in Fig. 23.2.
2. Similarly, soils B and C are also plotted on the chart, according to the proportion of gravel, sand, and fines, as points B and C.
3. The desired gradation of soil to be used for mechanical stabilization is decided in terms of percentages of gravel, sand, and fines using Eq. (23.1), or by any other suitable method.
4. This soil with the desired gradation is also plotted on the triangular chart as point D, based on its gravel, sand, and fines content.

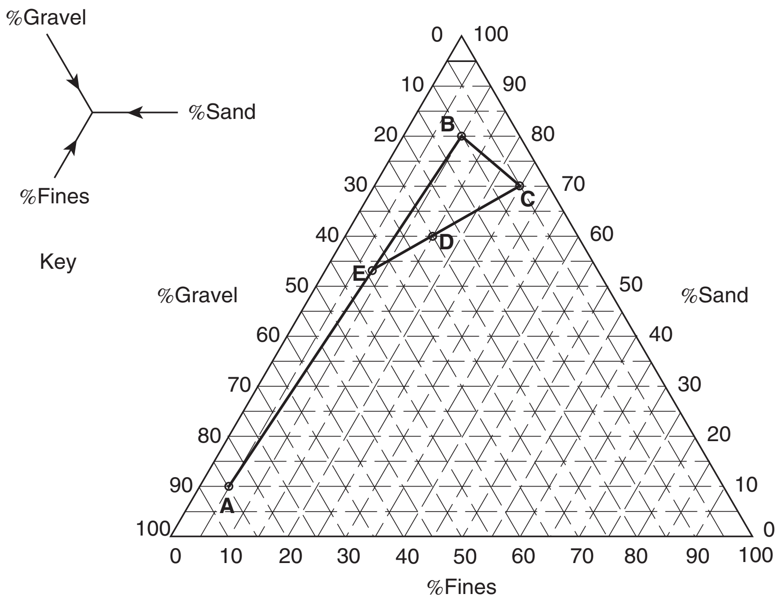

Figure 23.2 Proportioning by triangular chart.

Table 23.1 Example soils for triangular chart method

S. No.	*Borrow Area*		
	A	*B*	*C*
%Gravel	85	10	5
%Sand	10	80	70
%Fines	5	10	25

5. Points A, B, C, and D are joined by straight lines. Line CD is now extended to intersect line AB at E. The lengths of the lines are measured.
6. Now, the percentage of soil A to be used for mechanical stabilization to achieve the desired gradation is given by

$$p_A = \frac{BE}{AB} \times \frac{CD}{CE} \times 100 \tag{23.2}$$

7. Similarly, the percentages of soils B and C are given by Eq. (23.3) and Eq. (23.4), respectively,

$$p_B = \frac{AE}{AB} \times \frac{CD}{CE} \times 100 \tag{23.3}$$

$$p_C = \frac{DE}{CE} \times 100 \tag{23.4}$$

Rothfutch Method

The limitation of the triangular chart method is that proportioning is done based on the proportions of only gravel, sand, and fines. A more accurate method is to consider the complete grain-size distribution of each borrow soil. This is done in the Rothfutch method. The following is the procedure for proportioning in this method:

1. The desired gradation is first decided based on Eq. (23.1) or any other method. Unlike the triangular chart method, the complete grain-size distribution is determined by substituting a given particle size for d in Eq. (23.1),

and the maximum particle size for D and using a gradation index n = 0.5, the grain-size distribution is determined and shown in Table 23.2.

The particle size and percent finer are plotted as shown in Fig. 23.3, which gives the curve 1-2-3-4-5-6-7. However, the curve is converted into a straight line by changing the scale on the x-axis, and thus, points 2–6 are relocated to 2′–6′, as shown in Fig. 23.3, for the changed scale of the x-axis.

2. Now the grain-size distribution of the three borrow soils is plotted to the changed scale of the x-axis, and the balancing straight lines A, B, and C are drawn as shown in Fig. 23.3.
3. The zero end of balancing line A (point 8) is joined to the 100% end (point 10) of line B by a straight line. The point of intersection of this line 8–10 with the desired gradation line is marked as X.
4. Similarly, the zero end of balancing line B (point 9) is joined to the 100% end (point 12) of line C by a straight line. The point of intersection of this line 9–12 with the desired gradation line is marked as Y.
5. The proportion of borrow soils A, B, and C to achieve the desired gradation are determined based on the % finer corresponding to points X and Y, as shown in Fig. 23.3. Thus, for the given borrow soils, the proportions are A = 63.0%, B = 26.3%, and C = 10.7%.

Table 23.2 Desired gradation for proportioning

S. No.	*Particle Size (mm)*	*Cumulative Percent Finer*
1.	0.075	4.3
2.	0.425	10.3
3.	2	22.4
4.	4.75	34.5
5.	10	50.0
6.	20	70.7
7.	40	100.0

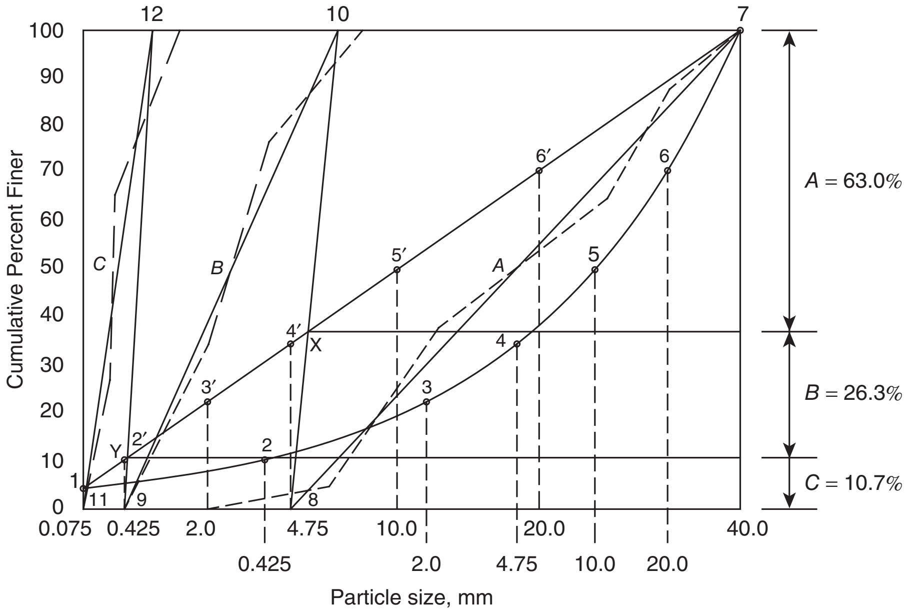

Figure 23.3 Proportioning in Rothfutch method.

23.5 Sand Compaction Piles

The principle of sand compaction piles consists of driving a hollow steel pipe up to the required depth, with its bottom closed with a collapsible plate, and filling the voids so created by compacted sand. After filling the pipe with sand, the pipe is withdrawn. During withdrawal, the bottom plate, which is of collapsible type, is opened and pressure is also applied against the sand inside the pipe. The sand backfills the voids created during pile driving.

Sand compaction piles are effective for compacting loose sand and gravel, especially below groundwater table, where the soil is saturated. The method is also effective in silty soils, above groundwater table, because of the displacement caused by pile driving.

Compaction of *in-situ* soil is achieved by the vibrations during pile driving and by the displacement caused in soils by the closed bottom and the casing. The diameter of the compacted zone around each pile is 7–12d, where d is the pile diameter. The size of the compacted zone increases with the increase in the initial relative density of the *in-situ* soil. It is usually possible to compact the *in-situ* soil to a relative density of 75%–80%. The method is economical for relatively small areas, compared with other soil improvement techniques. The maximum economical depth is about 20 m. The spacing of piles depends on site conditions but varies from 1.2 to 1.5 m. The sand used in sand compaction piles should have %fines < 15 and %clay < 3.

23.6 Blasting

Blasting is used to densify cohesionless soils at some depth below the ground surface.

23.6.1 Principle

In this method, a certain quantity of explosive charge is buried at a certain depth of a cohesionless soil and then detonated. The shock waves produced by the blasting cause densification.

23.6.2 Procedure

A casing pipe of 7.5- to 10-cm diameter is driven to the required depth in the soil. Sticks of dynamite and electric detonator are wrapped in waterproof bundles and lowered through the casing, as shown in Fig. 23.4.

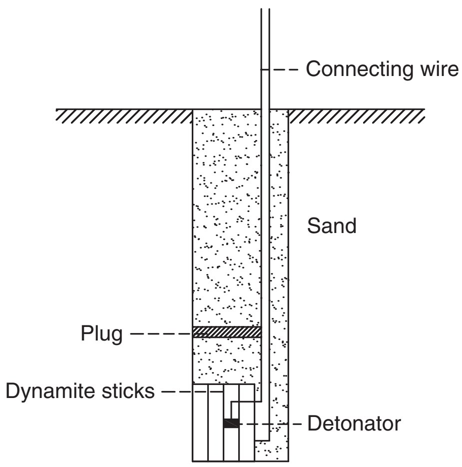

Figure 23.4 Blasting technique (schematic).

The casing is withdrawn and a wad of paper or wood is placed against the charge of explosives to protect it from misfire. The hole is backfilled with sand to obtain the full force of the blast. The circuit is closed to fire the charge by the following process:

1. A series of holes are thus detonated in succession and the large-diameter holes formed by lateral displacement after blasting are backfilled. Usually, the explosives are arranged in the form of a horizontal grid at a spacing of 3–8 m, but spacing less than 3 m should be avoided.
2. If the depth of stratum to be densified is less than 10 m, explosives are placed in one tier only below the depth, approximately at 2/3 point from the top. If the depth to be densified is more than 10 m, more than one tier of explosives should be planned.
3. Successive blasts of small charges at appropriate spacing are generally more effective than a single large blast.
4. The uppermost portion of the stratum may not be effectively densified by blasting, and this may be done by rollers. In multi-tiered blasts, the blasts should be timed from bottom to top. The amount of charge required is approximately given by

$$W = 164CR^3 \tag{23.5}$$

where W is the weight of explosive in kilograms, C the coefficient = 0.0025 for a 60% detonator, and R the radius of influence in meters. Charge of mass < 2 kgf to more than 30 kgf have been used.

23.6.3 Advantages

The main advantages of the blasting method are:

1. Blasting technique involves less time, labor, and expense.
2. No special equipment is required.
3. It can be successfully used to densify soil at large depth up to 20 m. Relative density of the order of 70%–80% can be achieved.

23.6.4 Disadvantages

Following are the disadvantages of blasting method:

1. There is no proper control over the densification process. Densification is not uniform in either the vertical (depth) or the lateral direction.
2. The method is unsuitable for fine-grained soils.
3. Maximum compaction in blasting occurs only when the soil is dry or saturated. In partially saturated soils, less densification is achieved due to capillary tension between soil grains.
4. Blasting may cause adverse effects on adjacent structures and hence can be used only in isolated sites or by maintaining the required minimum distance from adjacent structures.

23.7 Dynamic Compaction

Dynamic compaction consists of raising and dropping a heavy tamper onto the ground surface to compact the underlying soil deposits. The weight of the tamper, consisting of steel or concrete blocks, varying from 10 to 20 t, is dropped from a height of 10–25 m using special lifting equipment. Specialist lifting frames with quick-release mechanisms have been utilized to drop weights up to 50 t or even 170 t. Weights are made using a toughened steel plate, box-steel, and concrete, or suitably reinforced mass concrete where durability is the prime requirement. Dynamic compaction is many times referred to as dynamic consolidation, but the name does not qualify for this method, because dynamic compaction is a short duration method affected by dynamic loads. Consolidation on the other hand occurs under static loads by expulsion of pore water and takes very long time periods of a few months to years.

Impacts are applied at a grid spacing of 3–8 m by applying 5–15 blows in each pass, with a total of 2–8 passes, depending on the type of soil and its *in-situ* density. The ground can be improved by densification for depths of 10–30 m depending on the weight and the height of fall. Dynamic compaction is used for large projects such as roads, highways, railways, airport runways, gas or oil storage tanks, ports, etc.

The advent of large crawler cranes has led to further development of the method using high-energy tamping levels. In more advanced cranes, the whole work cycles are automated. The crane is controlled by a data processing unit that plots for each compaction point its location, number, weight size, drop height, number of blows, and measurement of imprint achieved.

23.7.1 Mechanism of Dynamic Compaction

Dynamic compaction strengthens weak soils by controlled high-energy tamping. The reaction of soils during dynamic compaction treatment depends on soil type and energy input. For better results of dynamic compaction, a comprehensive understanding of soil, that needs improvement, is essential. Dynamic compaction results in significant improvement to substantial depth economically, when compared with other ground improvement methods. However, care must be exercised for the treatment of soils with significant silt content, particularly below the water table. The mechanism of dynamic compaction for granular and cohesive soils is discussed as follows:

1. **Granular soils:** In dry granular materials, tamping improves engineering properties by physical displacement of particles, and the low-frequency excitation reduces the void ratio and increases the relative density to some extent, to provide improved load-bearing and enhanced settlement characteristics. When dynamic compaction is applied to fills consisting of coarse-grained soils, it results in the formation of a hard plug, as shown in Fig. 23.5, which inhibits penetration of stress impulses to the deeper layers and is very useful in providing superior settlement performance beneath isolated foundation bases.
2. **Cohesive soils:** Where the soils occur above the water table, the clays tend to be of relatively low moisture content and even a small reduction in volume by dynamic compaction can result in significant improvement in the bearing capacity. The drainage path is also relatively short. As such, treatment is relatively straightforward and rapid. For predominantly clay-type fill materials above the water table, the clay lumps can be considered as large weak particles of almost granular response. However, the major improvement is achieved by collapsing voids to provide a more intact structure. Clearly, the strength of the lumps and the sensitivity of the clay are of paramount importance in such soils.

The following factors are responsible to reduce the effectiveness of dynamic compaction in cohesive soils below the water table:

1. Development of heave due to tamping even in adjacent grids.
2. Quick development of excess pore pressures.
3. Long recovery time between successive passes and prolonged periods of treatment.
4. Prolonged time after completion of treatment for development of full soil strength.
5. Requirement of a larger number of low-energy impacts.

These factors virtually inhibit the usefulness and applicability of dynamic compaction for cohesive soils below the water table, making other ground improvement methods, such as vibro-compaction and vibro-replacement, more effective, successful, and economical. It is, therefore, desirable that dynamic compaction in cohesive soil deposits, if adopted, be carried in the driest season of the year, when the water table is deep.

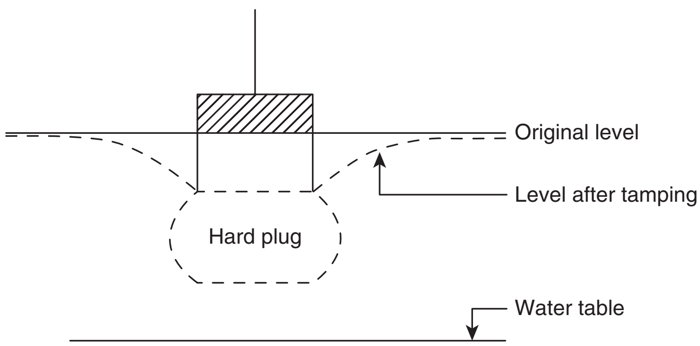

Figure 23.5 Dynamic compaction of coarse-grained soils.

23.7.2 Design Principles

For dynamic compaction, the ground is considered in three layers. The bottom layer is compacted by the first tamping pass at a relatively wide spacing and a suitable number of drops from the full-height capability of the crane. The middle layer is then treated by an intermediate grid, usually at the mid-point of the first pass or half the initial grid, with a lesser number of drops and reduced drop height. The surface layer is then compacted by a continual tamp of a small number of drops from low height in a continuous pattern, without spacing the grids.

Detailed measurement of imprint (Fig. 23.6) volume and surrounding heave or drawdown effect permits comparison of overall volumetric change with increasing energy input, and the process is known as the Shape test. Energy input beyond which no further improvement can practically be achieved is known as threshold energy. Threshold energy is sometimes exceeded deliberately, causing remolding and dilation of the soil, and this is known as over-tamping. The zone of major improvement in dynamic compaction varies from 1/2 to 2/3 of the effective depth of influence, which is the maximum depth at which significant improvement is measurable. The depth of improved ground may be obtained from

$$D = \alpha\sqrt{WH} \tag{23.6}$$

where D is the depth of improvement in meters, α the emprical coefficient is 0.5 for granular soils, W the weight of tamper in tons, and H the height of fall of the tamper in meters.

23.7.3 Occurrence of Liquefaction

When granular materials extend below the water table, a high proportion of the dynamic impulse is transferred to the pore water, which, after a suitable number of surface impacts, eventually rises due to pressure to a sufficient level to induce liquefaction. Low-frequency vibrations, caused by further stress impulses, will then reorganize the particles into a denser state. A vibrational acceleration in excess of 0.5 g may be necessary to achieve such a densification effect. Dissipation of the pore water pressures, in conjunction with the effective surcharge of the liquefied layer by the soils above, results in a further increase in relative density over a relatively short period of time. This can vary from 1 to 2 days for well-graded sand and gravel to 1–2 weeks for sandy silts.

Provision has to be made for this recovery time in dynamic compaction after initial passes and development of liquefaction so that the excess pore pressure is relieved and the soil gains density and strength. In other methods,

Figure 23.6 Dynamic compaction.

liquefaction is avoided and the treatment is designed to provide compaction by displacement without dilation or high-excess pore pressures by using a smaller number of drops from a lower drop height.

23.7.4 Applications and Suitability

The method is applied in isolated sites, which are away from the developed areas, in view of the impact of the dropping weights on the stability of existing structures. The method is found to be most effective for coarse-grained soils with <35% silt, which show a quick response to the treatment. The particles come to a closer packing and the high permeability permits quick dissipation of pore water pressures developed by impact. The method is also applicable for partially saturated semi-pervious deposits with < 25% clay and plasticity index (PI) < 8, having water content less than plastic limit. For saturated semi-pervious deposits, it takes several days to weeks for the reduction of excess pore pressures, requiring rest periods between successive passes. The method is unsuitable for saturated impervious soils.

23.8 Preloading

Expulsion of water from the pores causes consolidation of the soil, thereby resulting in buildup of shear strength and substantially reduced final settlements of foundations. This is achieved by preloading or pre-compression of the subsoil by subjecting the area to a preload. Preloading consists of placing a temporary surcharge on the ground prior to the construction of the planned structure. Preloading is generally carried out in stages to allow gradual buildup of soil strength, enabling it to safely support further stages of preload.

In the case of a building, the surcharge would be normally equivalent or higher than the expected bearing pressure. In the case of an embankment, it can be overbuilt prior to final shaping and commissioning to more rapidly achieve the level of settlements expected in the actual design. The surcharge generally consists of earth fill or any suitable material. In cohesionless soils, such as sand and gravels, lowering the groundwater table may provide an alternative means of increasing effective vertical stresses.

The preloading method is the most advantageous for improving soft cohesive soils, although it can be applied to all types of soils. The process can be speeded up with vertical drains, and in the case of relatively impermeable soils, with a horizontal drainage layer at the original ground surface. Subsoils exhibiting high secondary consolidation characteristics may not be amenable to the required degree of improvement by the preloading method. Removal of water from pore spaces has also been carried out by application of electric current to the subsoil, the process being known as electro-osmosis.

23.9 Sand Drains

The use of vertical sand drains was first proposed in 1925, and patented in 1926, by Daniel D. Moran. In early applications of vertical drains, sand drains consisting of bore holes filled with sand were used. The holes are formed by driving, jetting, or augering and would typically have diameters of 20–45 cm, spaced 1.5–6 m apart. Sand drains originally installed had generally a relatively large diameter, 0.4–0.6 m. To facilitate construction, minimize sand waste, and ensure continuity of the drain, the sand may be prepacked in a fabric sock, such as sand wick drain, which has a diameter of 6.5 cm. Later on, small-diameter sand drains came into use, for example, “sand wicks,” 0.05 m in diameter, and “fabridrains” – also called “sand pack drains” – 0.12 m in diameter. Figure 23.7 shows a typical sand drain system.

Vertical drains are installed to accelerate settlement and gain in strength of soft cohesive soils. In the absence of vertical drains, bearing failure may occur during placement of the surcharge fill used for preloading, and then settlement of clay may extend over many years.

Preloading with vertical drains has become an economical alternative to deep foundations or other methods of ground improvement and highly efficient drain installation methods have now been developed. Vertical drains are also advantageous in the construction of permanent fills, such as highway embankments on soft ground. Basic design principles are the same whether the surface is permanent or temporary.

Vertical drains accelerate primary consolidation alone, because significant water movement is associated with it. Secondary consolidation involves very small amounts of water movement draining from the soil. Secondary settlement is, therefore, not accelerated by vertical drains.

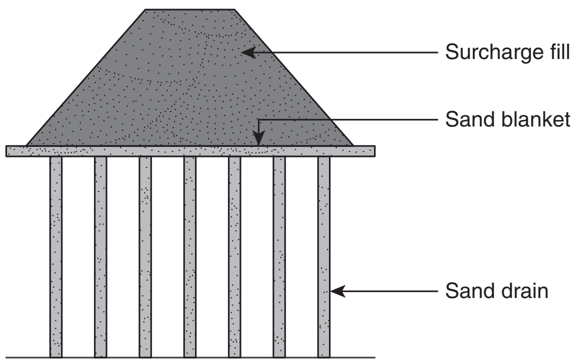

Figure 23.7 Sand drains.

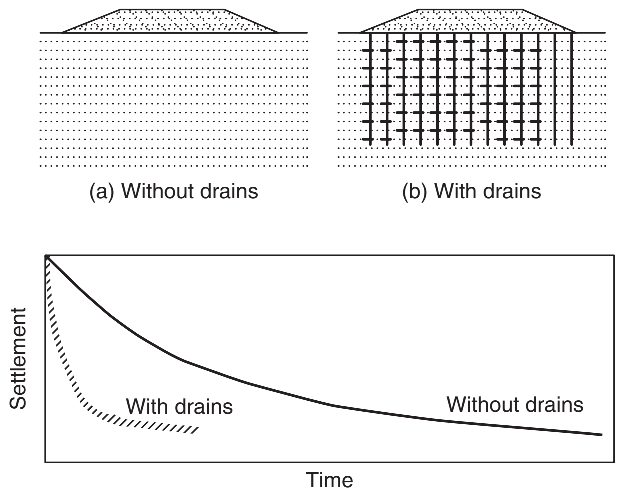

Figure 23.8 Vertical drains accelerate settlements but do not reduce final settlement.

According to Rowe, only relatively impermeable soils with $C_v < 3 \times 10^{-7}$ m^2/s, potentially benefit from vertical drains. Soils which are more permeable will usually consolidate under surcharge at an acceptable rate, on their own.

Vertical drains are particularly effective where a clay deposit contains many thin horizontal sand or silt lenses. The major beneficial effect of preloading and vertical drains is illustrated in Fig. 23.8. Preloading reduces total post-construction and differential settlements and considerably reduces the size and cost of foundation.

23.10 Prefabricated Vertical Drains

Prefabricated vertical drains are fluted textile material in the shape of a strip to act as a vertical drainage face. They are an effective alternative to sand drains to enhance radial consolidation and accelerate pre-construction compression and settlement of soft cohesive soils.

23.10.1 Principle

Prefabricated vertical drains in the shape of a band, strip, or wick are used as a more effective substitute to sand drains. They consist of a central core wrapped by a filter layer. The permeability of the filter will be much greater than that of the surrounding soil. The filter should effectively retain the soil particle while allowing the entry of water and should be strong enough to resist the installation stresses.

23.10.2 Band Drain or Wick Drain

The first type of band drains, also known as wick drains, introduced in the market was invented in Sweden by Walter Kjellman at the Swedish Geotechnical Institute. These drains, named cardboard wicks (Kjellman, 1947), were made of two cardboard sheets glued together with an external cross section of 100 mm × 3 mm and included 10 longitudinal internal channels, 3 mm in width and 1 mm in thickness [Fig. 23.9(a)]. Band drains, shown in Figs. 23.9(b) to (d) proved to be both economically and technically superior to the conventional sand drains. Geodrain, developed at the Swedish Geotechnical Institute, consists of a core of plastic material surrounded by a filter sleeve with an external cross section of 95 mm × 4 mm, that is, nearly the same dimensions as the cardboard wick. The filter sleeve is made of synthetic material. The cardboard drains have now become obsolete, replaced by other types of band drains made of geotextiles.

23.10.3 Geosynthetic Drain

The first geosynthetic drain, called strip drain, was developed by the Swedish Geotechnical Institute, which was made of cardboard with internal ducts. This was later superseded by fluted polyvinyl chloride (PVC) drains. Today, there are more than 50 varieties of strip drains in the market, mostly of composite construction. Strip drains consist of a corrugated or studded inner core wrapped in a filter fabric made of non-woven geotextile. They are generally about 10 cm wide and 2–6-mm thick.

23.10.4 Equivalent Diameter

In the radial consolidation theory, the drain is assumed to be of circular cross section. Hence, an equivalent diameter is to be calculated for strip drains. Based on the equivalent void area concept (Koerner, 1986), the equivalent diameter of a strip drain (d_e) is given by

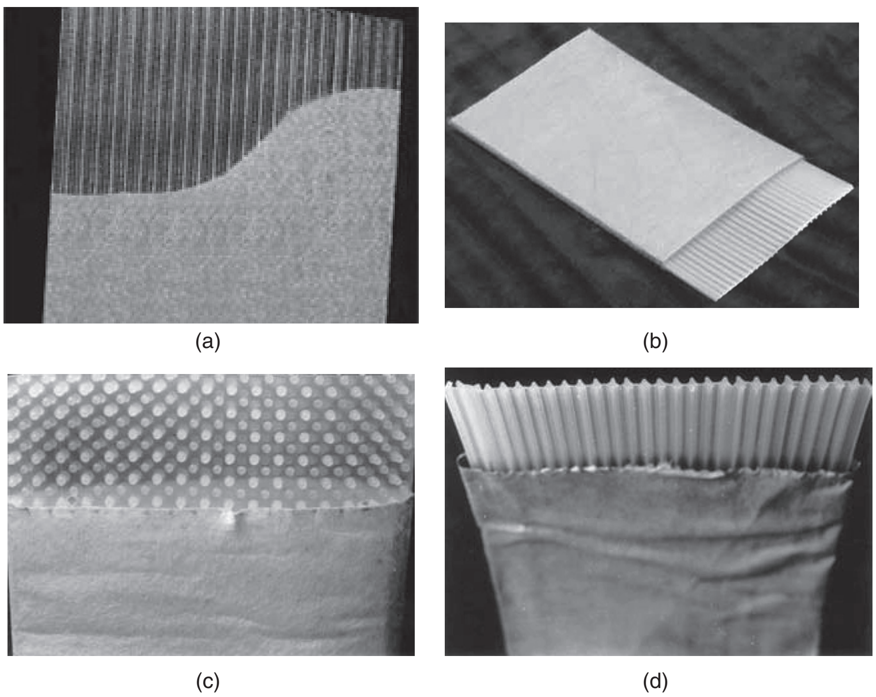

Figure 23.9 Cardboard wick drain and band drains: (a) Cardboard wick drain, (b) band drain with central core without filter sleeve, (c) and (d) band drains with central core surrounded by filter sleeve.

$$d_e = \sqrt{\frac{4B\ t\ n_d}{\pi n_s^2}} \tag{23.7}$$

where B is the width of the strip drain, t the thickness of the strip drain, n_d the void area per unit cross-sectional area of the strip drain, and n_s the porosity of the strip drain.

Based on the equal circumference concept (Hansloo, 1979), the equivalent diameter of a strip drain (d_e) is given by

$$d_e = \frac{2(B+t)}{\pi} \tag{23.8}$$

Equation (23.8) gives a conservative value of d_e compared with that given by Eq. (23.7).

23.10.5 Construction

Sites in need of vertical drainage often have a low bearing capacity and may need to be reinforced to enable the installation machinery to enter the site without risk of soil failure. A sand layer 0.3–0.5-m thick, is placed in advance on the ground surface before drain installation. It serves the dual purposes as a working platform to safeguard the installation work as well as a drainage layer. A geotextile may be used between the clay and the sand layer to prevent mixing of the two layers and loss of the sand layer. Figure 23.10 shows vertical drains installed for a highway project.

The drains are usually installed in an equilateral triangular or square pattern. Irrespective of which drain pattern is chosen, the number of drains required to achieve a certain rate of consolidation will be more or less the same. However, the equilateral triangular pattern is most optimal and preferable.

After completion of drain installation, the drained area is loaded with a fill embankment. Stability along the edge of the fill embankment should be ensured. The cost of loading berms may add considerably to the total cost of a vertical drainage project. Loading berms are no longer required and are replaced by the vacuum method, which is an alternative to preloading by the use of a fill embankment.

The drains are installed inside a steel mandrel, which protects the drains from being damaged. Installation can be done by static or dynamic methods. In the first case, the mandrel with the drain inside is pushed into the soil by static pressure, while in the second case, it is driven into the soil by means of a gravity hammer or vibratory driver. The method of installation – static or dynamic – does not seem to affect the efficiency of the drainage system. Dynamic methods should, however, be avoided wherever disturbance effects, usually evidenced by excess pore water pressure being built up during installation, may affect stability.

Before the drains are inserted into the soil, they must be provided with an anchor, which keeps the drains in position when the mandrel is withdrawn (Fig. 23.11). The anchor also prevents the soil from intruding into the

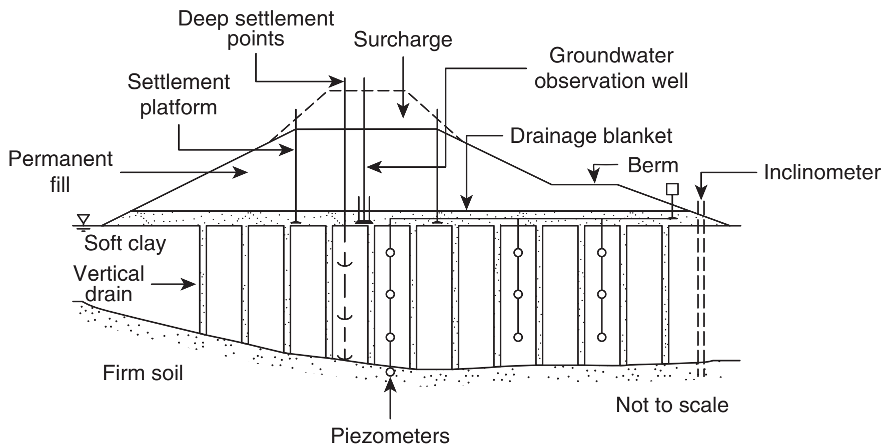

Figure 23.10 Prefabricated vertical drains for a highway embankment.

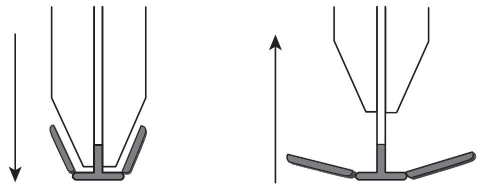

Figure 23.11 Anchor for band drains.

mandrel during installation (which may lock the drain to the mandrel by friction). After the drains are installed to the required depth, they are cut at the top of the drainage layer and a new anchor is attached to the bottom end of the drain for installing the next position. The mandrel should be stiff enough to ensure vertical drainage.

The smearing of surface of the drains during installation reduces their permeability and effectiveness of drainage. The stiffer the soil, the larger is the zone of influence (smear), and vice versa. The extent of the smear zone can be put equal to 3–4 times the cross-sectional area of the band drain.

23.10.6 Advantages and Disadvantages

The main advantages of synthetic drains compared with sand drains are as follows:

1. Easy and rapid installation is possible. In a typical installation of 15-m-long drains at a spacing of 1–2.5 m center-to-center (c/c), a placement rate of up to 375-m length of drain per hour is possible.
2. Synthetic drains are made of uniform material of consistent quality and can be easily stored and transported.
3. Synthetic drains require lighter equipment compared with the rigs used for sand drains.
4. The tensile strength of strips helps to preserve continuity.
5. Synthetic drains are more economical than sand drains. Ground improvement with synthetic drains cost only about one-fourth the cost of traditional sand drains.
6. Prefabricated vertical (PV) drains cause less disturbance to the soil.
7. The continuity of the drain is maintained more effectively compared with sand drains.

Some of the disadvantages of PV drains are as follows:

1. Ground settlement can cause folding or buckling of the PV drains and reduce their efficiency.
2. The effect of smear in the remolded soil portion near the drain reduces radial consolidation.
3. Clogging or siltation of the filter drain by fine soil particles can decrease the area of flow for water.

23.11 Stone Columns

Stone columns are columns constructed of highly densified, open-graded stone or sand that are used to support embankments, lightweight structures, and retaining structures in soils with a very low bearing capacity. The material used in the construction of stone columns is well-graded clean sand, gravel, or stones. In Japan, they are called sand compaction piles. The term "granular pile" is being advocated as it is more appropriate.

23.11.1 Mechanism of Load Transfer

Stone columns provide the primary functions of reinforcement and drainage and, in addition, improve the strength and deformation properties of soft soil. They increase the unit weight of *in-situ* soil by replacement, drain rapidly the excess pore pressures generated, act as strong and stiff elements in *in-situ* soils, and carry high shear stresses. Figure 23.12 shows the improvement in shear strength due to construction of stone columns.

Stone columns create a composite ground of lower overall compressibility and higher shear strength than the original *in-situ* soil. When loads are applied, the stone columns and the weak soil move downward, resulting in an

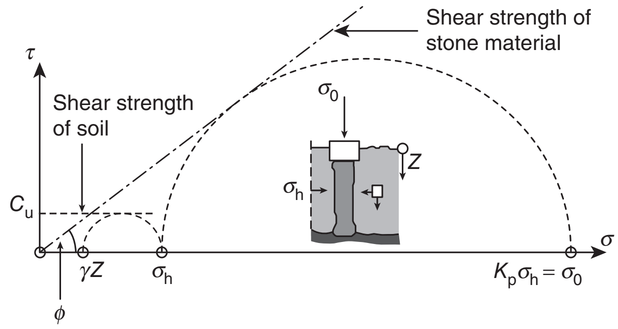

Figure 23.12 Mohr's circle for stone column–treated soil.

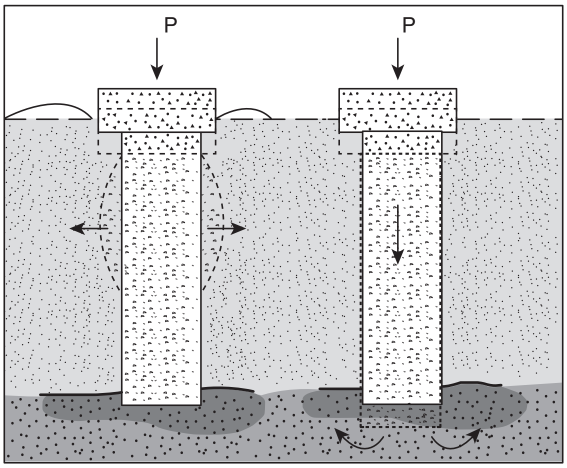

Figure 23.13 Brauns' failure mechanism in stone columns.

important concentration of stress within the stone columns. Load applied at the top of the stone column produces a bulge to a depth of about 2–3 diameters beneath the surface. This bulge in turn increases the lateral stress.

When the stone column is subjected to ultimate load, failure takes place (a) by bulging in the top one-third length of the column due to relatively small lateral support or (b) by punching of the toe of the column into the underlying soil, as shown in Fig. 23.13.

23.11.2 Design Principles

For design of stone columns, the effect of stone column construction is considered in terms of a parameter known as improvement factor. It is defined as the ratio of compression modulus of treated ground to that of untreated ground. Figure 23.14 shows the improvement factor as a function of ratio of the stone column area and the area being treated by the column as a function of angle of internal friction of the stone column. The improvement factor indicates the factor by which the compression modulus increases for a grid of stone columns and the extent by which the settlement of the foundation will be reduced.

The design of stone columns can be done using the design curves similar to the ones shown in Fig. 23.14. The design assumes that the stone column material is incompressible, and Fig. 23.15 allows an adjustment to be made for this by plotting a fictitious area ratio, which has to be added to the actual area ratio.

Practical design charts for evaluation of settlement of footings supported on stone column treated ground have been presented by Priebe (1995), considering the load distribution as well as reduced lateral support on columns situated underneath footing edges. The settlement of the footings supported on stone columns can be determined by using the design chart shown in Fig. 23.16.

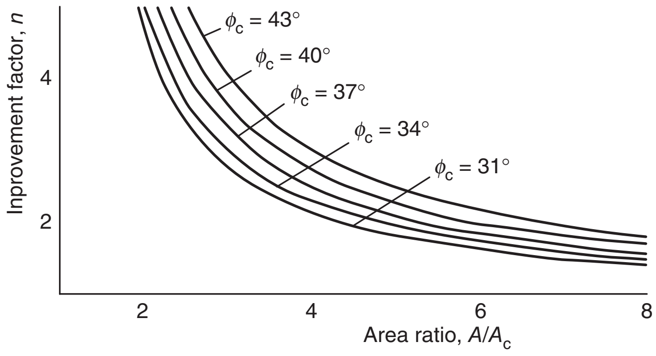

Figure 23.14 Improvement of compression modulus as a function of area ratio.

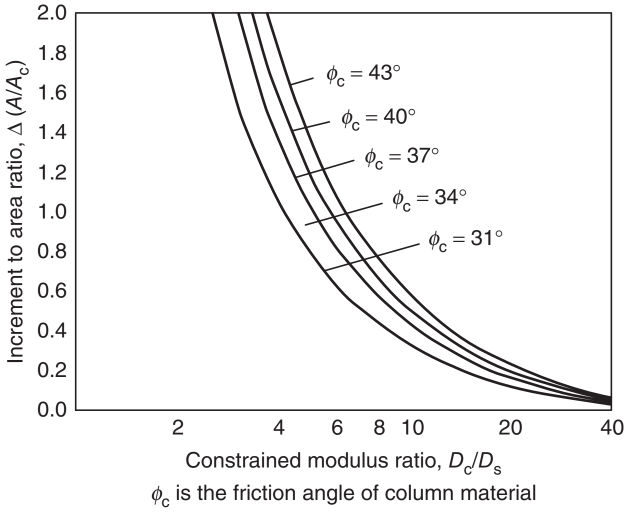

Figure 23.15 Increment to area ratio as a function of modulus ratio.

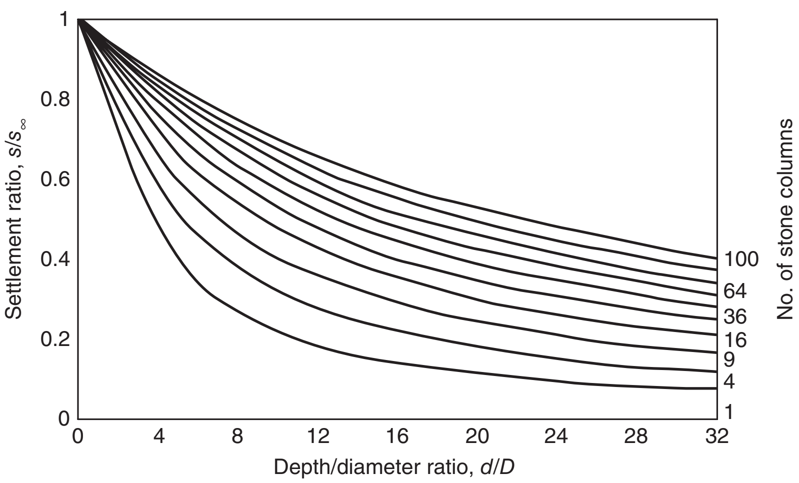

Figure 23.16 Evaluation of settlements for isolated footings supported on stone column-improved ground.

23.11.3 Construction of Stone Columns

Stone columns can be installed by any of the following methods depending on the site soil, available equipment, and proven applicability:

1. Vibro-compaction method.
2. Vibro-replacement method.
3. Vibro-composer method.
4. Cased bore hole method.
5. Geotextile-coated stone columns.

23.11.3.1 Vibro-Compaction Method

Vibro-compaction is the oldest dynamic deep compaction method in existence, developed by Johann Keller Company in 1936, and is most effective for non-cohesive soils. This method is used to improve the density of cohesionless soils. The range of soil types that is treated by vibro-compaction and vibro-replacement is given in Fig. 23.17. The effectiveness of the method is based on the fact that cohesionless soils are well-compacted by vibration. Sands and gravels bearing negligible cohesion are compatible with vibro-compaction. The suitable proportion of gravel is 0%–80% and sand 20%–100%, with zero percent fines. The silt (grain size < 0.06 mm) percentage of such soils should be less than 10% for ideal performance. Depths down to 65 m have been improved so far by vibro-compaction.

The equipment used in the method is a vibro-flot, shown in Fig. 23.18. It consists of three main parts: a vibrator, extension tubes, and a supporting crane. A vibro-flot, also called vibratory probe, is a cylindrical tube containing air or water jets at the top and bottom and is equipped with a rotating eccentric weight which develops horizontal vibratory motion. The vibro-flot is sunk into the soil using bottom jets and is then raised in small increments, during which the surrounding material is compacted by vibration. The soil is liquefied locally around the vibrator during compaction, and new material can be added either at the surface or through the extension tube and the vibrator.

The spacing of vibro-compaction probes depends on the soil being treated, the degree of densification required, the type of vibrator being used, and production rates. Areas treated per probe vary commonly between 6 and 20 m^2.

The thickness of the liquefied zone is 0.30–0.55 m, and it decreases with increasing permeability and particle size. The improvement of the compaction beyond a radial distance of 2.5 m from the vibrator is usually insignificant,

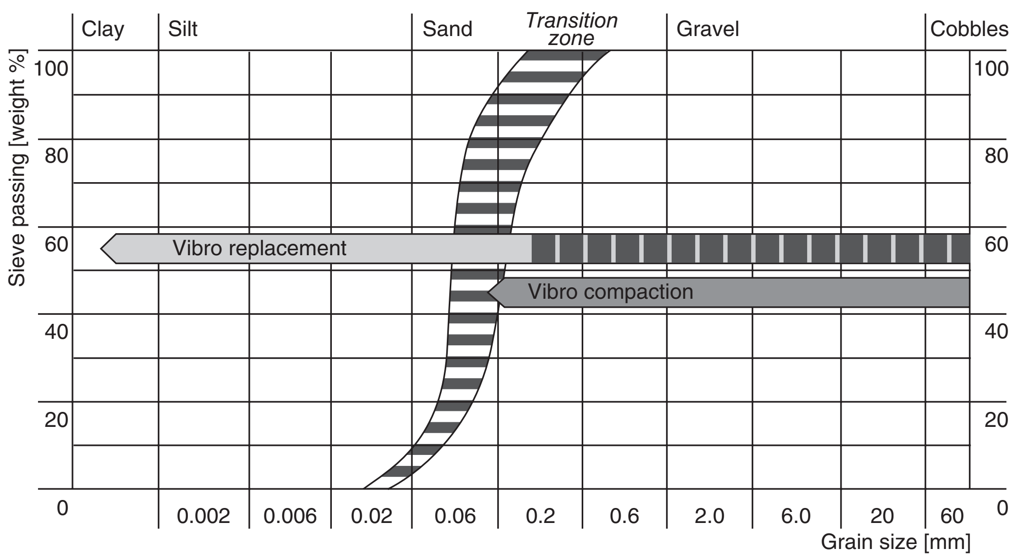

Figure 23.17 Grain-size distribution suitable for vibro-compaction and vibro-replacement. (After Keller, 1936)

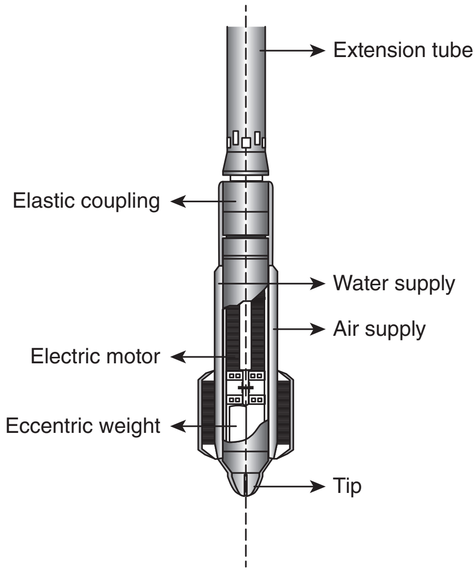

Figure 23.18 Vibratory probe for vibro-compaction and vibro-replacement.

regardless of the duration of compaction. The diameter of the stone columns is generally larger at the surface than at the base. The time required for constructing each stone column is about 10–30 min. The main disadvantage of the method is often the difficulty in disposing of the excess water during installation. Vibro-compaction is unsuitable in water-saturated sands with high silt content as the liquefied soil is not effectively densified within a reasonable time.

To increase the performance of the vibro-system, multiple vibrators may be applied on one base machine. For example, a barge with a 120- to 150-t crane with four vibrators was used for the Seabird project in India (Keller, 2002). Alternatively a special frame was constructed on a barge suspending five vibrators (Keller, 1997).

The degree of improvement will depend on many more factors including soil conditions, type of equipment, procedures adopted, and skills of the site staff. The effectiveness of vibro-compaction can be readily checked using standard penetration tests or cone penetration tests. Comparisons can be made between pre- and post-compaction testing, and care should be taken to ensure that the same techniques of testing are used. Control of performance is best achieved by using a standardized procedure, established at the pre-construction trial, such as predetermined lifts of the vibrator at predetermined time intervals and/or predetermined power consumptions.

23.11.3.2 Vibro-Replacement Method

To overcome the limitations of the vibro-compaction method, the vibro-replacement method was developed. In this method, the vibrator is inserted into the soil without the aid of simultaneously flushing in water. After the vibrator is lifted, the temporarily stable cylindrical cavity is filled with a coarse material, section by section. The coarse material is then compacted by repetitive use of the vibrator. This vibro-replacement procedure, known as the conventional dry method, is used for construction of stone columns. Vibro-replacement is widely used in Europe to improve a wide range of treatable weak natural soils and man-made fills for safe and economic construction of residential and light commercial and industrial structures.

The equipment developed for the vibro-compaction and vibro-replacement processes comprises the following four basic elements:

1. Vibrator, which is elastically suspended from extension tubes with air or water jetting systems.
2. Crane or base machine, which supports the vibrator and extension tubes.
3. Stone delivery system used in vibro-replacement.
4. Control and verification devices.

Acceleration of consolidation, reduction of consolidation time and compressibility, and increase of the load-bearing capacity and shear strength as well as reduction of overall settlement in soft fine-grained soils are the goals of the vibro-column installation.

In the conventional dry method, the vibrator is used to displace the surrounding soil laterally. The crushed stone is pressed laterally into the soil during both the cavity-filling stage and the compaction stage. This produces stone columns that are tightly interlocked with the surrounding soil. Groups of columns created in this manner can be used to support large loads.

The disadvantage with the conventional dry method in cohesive soils with a high water content is the possible cavity collapse. This can be overcome by dry vibro-replacement method, in which the bottom feed vibrators introduce the stones and compressed air through the vibrator tip during withdrawal of the vibrator, preventing cavity collapse, as shown in Fig. 23.19.

Vibro-stone columns are not suitable in soft clays or silts or fine sands with very low undrained cohesion because the lateral support is too small. However, vibro-stone columns have been installed successfully in soil with undrained cohesion in the range of 5 kPa < c_u < 15 kPa. The vibro-replacement method is also useful in reducing the liquefaction potential.

23.11.3.3 Vibro-Composer Method

The vibro-composer method involves driving the casing pipe to the desired depth using a heavy vertical vibratory hammer, located at the top of the pipe. The casing is filled with a specified volume of sand and the casing is then repeatedly extracted and partially redriven using the vibratory hammer, starting from the bottom. The process is repeated until a fully penetrating compacted granular pile is formed. This method is popular in Japan and is used for improvement of soft clays in the presence of high groundwater levels. The installation procedure is illustrated in Fig. 23.20. The resulting pile is termed as sand compaction pile.

23.11.3.4 Cased Bore Hole Method

In this method, the piles are constructed by ramming granular materials into the prebored holes in stages using a heavy weight 1–2 t, falling freely from a height of 1–1.5 m. The piles constructed by this method are most commonly called rammed stone columns. These piles incorporate the additional benefits of heavy tamping as they, in effect, are preloaded. This method is a good substitute for vibratory compaction, considering its low cost. The method is, however, not applicable to sensitive soils due to disturbance and subsequent remolding. Figure 23.21 illustrates the installation procedure. A certain distance should be kept to existing buildings to limit settlements of new buildings due to vibration and drilling operations.

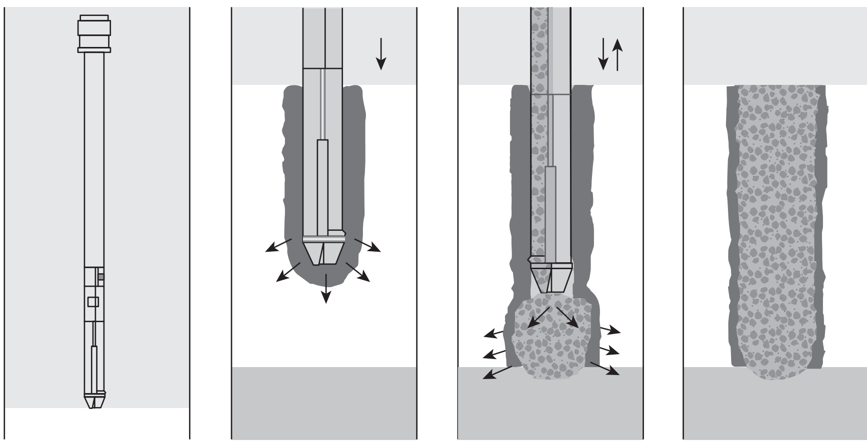

Figure 23.19 Vibro-replacement method: bottom feed.

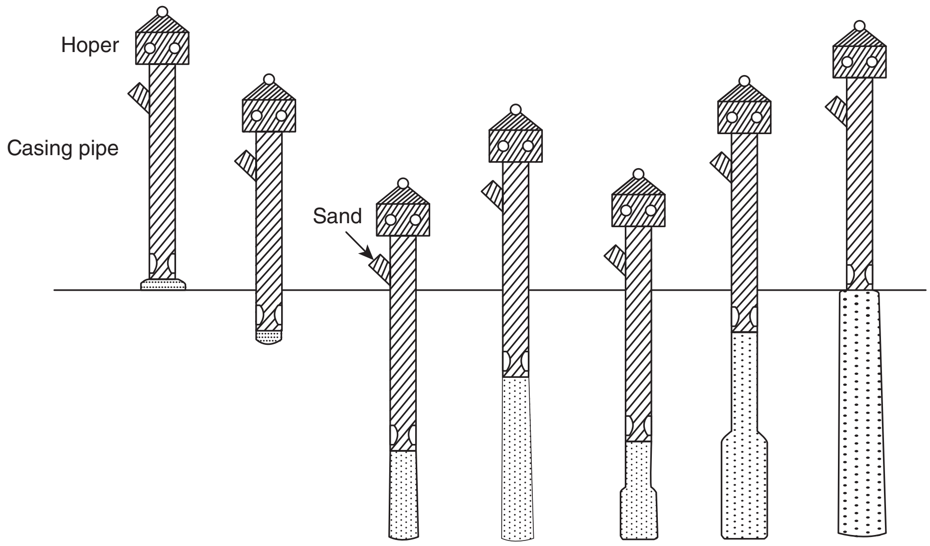

Figure 23.20 Vibro-composer method.

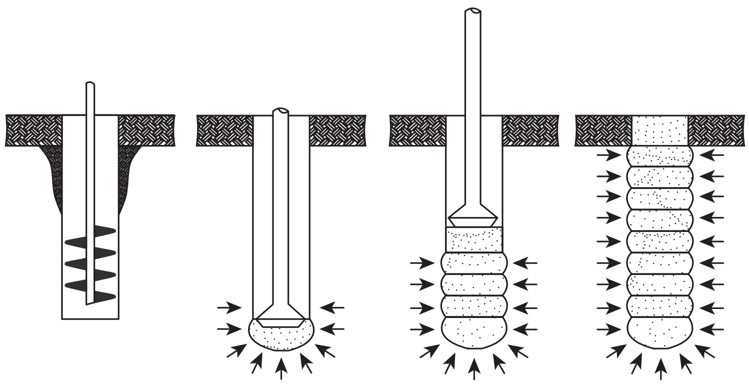

Figure 23.21 Cased bore hole method.

23.11.3.5 Geotextile-Coated Stone Columns

In very soft nearly liquid soils, vibro-replacement is not effective due to the lack of lateral support of the soil. A geotextile coating may be used around the column to ensure filter stability and to activate tensile forces to avoid lateral spreading of the column. This method, known as geotextile-coated column, was developed in 1992 and applied first in early 1993 for a dam project in Austria (Sondermann and Wehr, 2004).

Vibro-geotextile columns consist of a sand or stone core with a geotextile coating. The advantage of a vibro-geotextile column compared with other stone columns is the well-densified granular infill, resulting only in small settlements of the soil-column system.

The installation is usually performed in several steps so as not to damage the geotextile. First, a hole is created with the vibrator to the required depth and the vibrator is extracted. In the next step, the geotextile is mounted over the vibrator above the ground surface and, subsequently, the penetration is repeated with the geotextile to the same depth as before. On the way up, stones are filled and densified inside the geotextile like in the usual dry bottom feed process.

If the very soft layer extends over a limited depth, it is possible to build a vibro-stone column below this layer first, insert a vibro-geotextile column only in the very soft layer for economical reasons, and finish the upper part of the column as an ordinary vibro-stone column again.

23.11.4 Advantages and Applications

Stone columns are more advantageous than other ground improvement techniques because of their technical feasibility, low energy utilization, and cost-effectiveness. Stone columns provide an economical alternative to deep foundations in soft or weak and compressible soils. Installation of stone columns is relatively simple and requires less labor. Granular piles are also one of the most preferred choices to improve liquefaction resistance of loose sands and to minimize settlements after a seismic event.

23.12 Reinforced Earth

Soils which have almost negligible tensile strength can be strengthened by the inclusion of materials with high tensile strength. This mobilization of tensile strength is obtained by surface interaction between the soil and the reinforcement through friction and adhesion. The reinforced soil is obtained by placing extensible or inextensible materials such as metallic strips or polymeric reinforcement within the soil to obtain the requisite properties.

Modern applications of reinforced soil for construction of retaining walls were developed in France by Henry Vidal in 1963. The Vidal system, called reinforced earth, used metal strips for reinforcement, as shown in Fig. 23.22.

Reinforced soil structures fall, broadly, into the following three classes:

1. Mechanically stabilized earth (MSE) walls.
2. Reinforced slopes and embankments.
3. Reinforced foundations.

"Mechanically stabilized soil mass" is a generic term that includes reinforced fill, in which multiple layers of inclusions act as reinforcement, and multi-anchored soil mass, in which multiple layers of inclusions act as anchored tendons in soils placed as fill.

Reinforced earth is a constructed composite material and consists of alternate layers of compacted backfill and tensile reinforcing material. Both metals and geotextiles are used as reinforcements. The main features of reinforced earth are as follows:

1. The reinforcements carry only tension.
2. It is a composite material in which the soils and the reinforcements are built up in successive layers.

23.12.1 Principle of Reinforced Earth

The basic principle of reinforced earth is shown in Fig. 23.23. The reinforced earth results from the association of two components having different modulus of elasticity. A stress applied to the mass will cause strain in the soil, which will transmit tensile load to the reinforcing strips. The displacements are restrained in the direction of the reinforcing strips, causing the reinforced mass to behave like a cohesive anisotropic material. Hence, the concept of

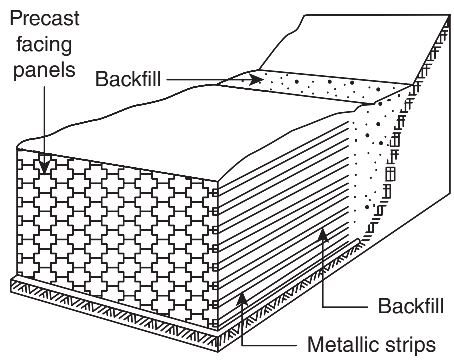

Figure 23.22 Henry Vidal's reinforced earth.

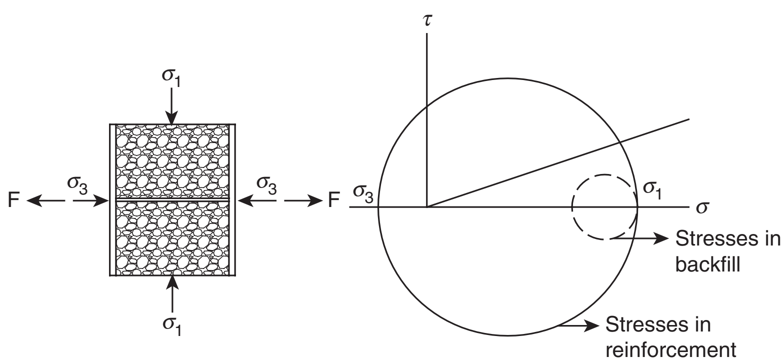

Figure 23.23 Principle of reinforced earth.

reinforced earth is based on frictional interaction between the earth and the reinforcement. It develops shear stress at the soil–reinforcement interface, given by

$$\tau = \frac{1}{2b}\frac{dT}{dl} \tag{23.9}$$

where τ is the shear stress at the soil–reinforcement interface at any distance l along the reinforcing strip, T the tensile force in the reinforcement, and b the width of the reinforcement.

23.12.2 Components of Reinforced Earth

Figure 23.24 shows the various components of reinforced earth and includes the following:

1. Reinforcement.
2. Backfill.
3. Facing units.
4. Leveling pad or footing.

23.12.2.1 Reinforcement

Reinforcing elements (inclusions) are placed in a soil mass to improve its mechanical properties. Soil reinforcement may be made with a number of materials as follows:

1. Woven geotextiles.
2. Polymer geogrids of polyethylene (usually uniaxial) and polypropylene (usually biaxial).
3. Polyester and fiberglass geogrids (often knitted or stitched at junctions) usually coated with a polymer such as polyethylene or PVC or with bitumen.
4. Steel strips (the original “reinforced earth”).
5. Welded wire mesh.

Galvanized steel strips are the most common type of reinforcement used. Each element is a thin strip of metal, 5–10-cm wide and up to 0.9-cm thick. Metal rods, wire grids, and geotextiles are also used as reinforcement.

Selection of reinforcement is made based on two criteria: (a) survivability during construction and (b) Strength requirement for stability. In some instances, considerations of survivability will dictate a more durable/stronger product than would be required by considering strength requirement alone.

23.12.2.2 Backfill

The backfill material used is the granular soil with less than 15% fines.

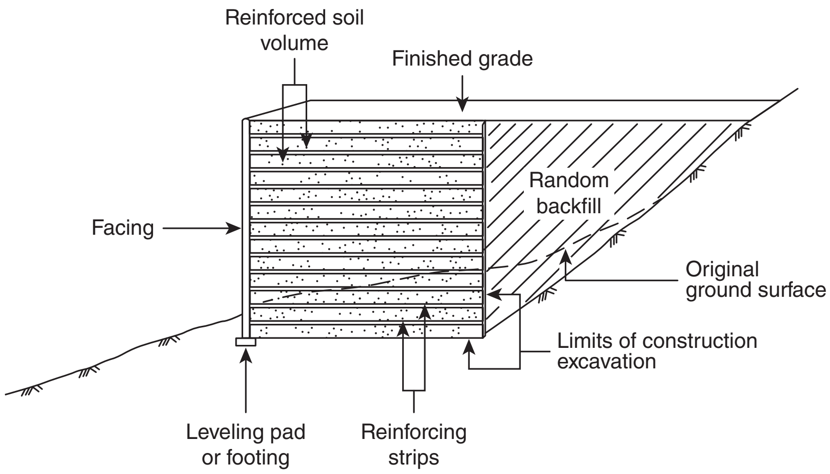

Figure 23.24 Components of reinforced earth.

23.12.2.3 Facing Units

It is necessary to provide some form of a barrier at the free boundary of a reinforced earth to retain the soil. These facing units must be strong enough to hold back the local soil and to allow fastenings to attach the reinforcement. Prefabricated units, which are small and light enough for easy transportation, and quick and easy installation, may be used as facing units. The prefabricated units are generally made from steel, aluminum, reinforced concrete, or plastic. The facing units require a small foundation on which they can be built. It consists of a trench filled with mass concrete similar to wall footings. Different types of geotextile-reinforced wall facings can be adopted, as shown in Fig. 23.25.

23.12.2.4 Levelling Pad or Footing

A cast-in-place or precast concrete levelling pad is placed at the foundation elevation for the erection of the facing elements for all reinforced fill structures with precast facing elements. The purpose of the pad is to serve as a guide for facing panel erection and not to act as a structural foundation support. The pad may be of 12.7-mm thickness by 30.5-mm width and should have a minimum 13.8 MPa compressive strength.

23.12.3 Stress Transfer Mechanism

Stresses are transferred between the soil and the reinforcement by friction and/or passive resistance, depending on reinforcement geometry.

23.12.3.1 Friction

In this mechanism, stresses are transferred from soil to reinforcement by shear along the interface. This is the dominant mechanism with linear and planar reinforcements (strips, rods, cables, nails, fabrics, and geotextiles sheets).

23.12.3.2 Passive Resistance

In this mechanism, stresses are transferred from soil to reinforcement by bearing between the transverse elements against the soil. This is the dominant mechanism for reinforcement containing a large number of transverse elements of composite inclusions such as bar mats, grids, and wire mesh.

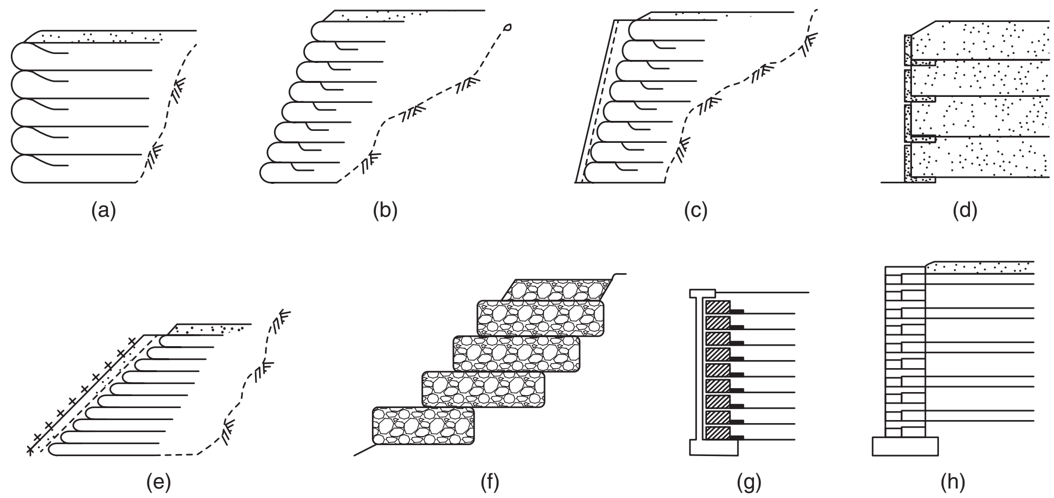

Figure 23.25 Types of geotextile-reinforced wall facing (FWHA, 1990): (a) Vertical geotextile facing, (b) vertical geotextile facing, (c) sloping gunite or structural facing, (d) vertical precast concrete element facing, (e) sloping soil and vegetation facing, (f) geotextile gabion, (g) vertical cast in situ concrete or masonry facing, and (h) vertical masonry facing.

23.12.4 Construction of Reinforced Fill Systems

Construction of reinforced soil systems is relatively simple and rapid. The construction sequence consists mainly of the following operations:

1. Preparation of the subgrade.
2. Placement of a leveling pad.
3. Erection of facing panels – placing the first row.
4. Compaction of reinforced fill.
5. Placement of reinforcing elements.
6. Placement of second and subsequent facing courses (segmental facings).
7. Installation of drainage.

Special skills or equipment are usually not required, and locally available labor can usually be used. The various steps in construction are described in detail in the following subsections.

23.12.4.1 Preparation of the Subgrade

This involves removal of unsuitable materials from the area to be occupied by the retaining structure. All organic matter, vegetation, slide debris, and other unstable materials are stripped off and the subgrade compacted, if required.

23.12.4.2 Placement of a Leveling Pad

A cast-in-place or precast concrete leveling pad is placed at the foundation elevation for the erection of the facing elements for all reinforced fill structures with precast facing elements. The purpose of the pad is to serve as a guide for facing panel erection and not to act as a structural foundation support. The pad may be of 12.7-mm thickness by 30.5-mm width and should have a minimum 13.8 MPa compressive strength.

Cast-in-place pads are cured a minimum of 12 h before facing panels are placed. Full-height precast facing elements may require a larger leveling pad to maintain alignment and provide temporary foundation support.

23.12.4.3 Erection of Facing Panels – Placing the First Row

Facings may consist of either precast concrete panels, or metal facing panels, or fully flexible wrap-type facings, including welded wire mesh, geotextiles, and geogrids. The erection of segmental facing panels and placement of the soil backfill proceed simultaneously.

Precast facing panels are set at a slight backward batter of about 1 in 48 toward the fill to balance minor outward movement of the facing elements during wall fill placement and compaction. When using full-height panels, the construction should be carefully controlled using additional bracing and larger face panel batter, where necessary.

The first row of panels are laid directly on the concrete leveling pad. Construction should always begin adjacent to any existing structure and proceed toward the open end of the wall of panels and the leveling pad.

The first row of panels are continuously braced until several layers of reinforcement and backfill have been placed to give initial support. Adjacent panels are clamped together to prevent individual panel displacement. When using full-height panels, initial bracing alignment and clamping should be done very carefully because small misalignments cannot be corrected in the construction of subsequent rows.

Most reinforced fill systems will use a variety of panel types on the same project to accommodate geometric and design requirements (geometric shape, size, finish, connection points).

23.12.4.4 Compaction of Reinforced Fill

Wall fill material should be placed and compacted at or within 2% dry of the optimum moisture content. If the reinforced fill is free-draining with less than 5% passing 75-μm sieve, water content of the fill may be within ±3% OMC.

Laying of reinforcement and compaction of backfill should be carried in the rear and middle portions behind the wall and bladed toward the front face. The construction equipment should not be in direct contact with the reinforcements because protective coatings and reinforcements can be damaged. Soil layers should be compacted up to or even slightly above the elevation of each level of reinforcement connections prior to placing that layer of reinforcing elements.

Large smooth drum vibratory rollers are generally used with 3–5 passes to obtain the desired compaction of the backfill except within the 0.9-m zone directly behind the facing elements or slope face. Sheep's foot rollers should not be used because of possible damage to the reinforcement. When compacting uniform medium-to-fine sands (in excess of 60% passing a 425-μm sieve), use a smooth drum static roller or lightweight (walk-behind) vibratory roller.

Small, single- or double-drum, walk-behind vibratory rollers or vibratory plate compactors should be used to compact backfill within 0.9 m of the wall or slope face. Placement of the reinforced backfill near the front should not lag behind the remainder of the structure by more than one lift.

Compaction control of the reinforced backfill should be performed on a regular basis during the entire construction project. A minimum frequency of one test within the reinforced soil zone every 0.6 m of wall height for every 30 m of wall is recommended.

23.12.4.5 Placement of Reinforcing Elements

Reinforcing elements are generally placed perpendicular to the back of the facing panel. For abutments and curved walls, it may be permissible to skew the reinforcements in either the horizontal or the vertical direction. In all cases, overlapping layers of reinforcements should be separated by a minimum 76-mm thickness of wall fill. Under no circumstances should adjacent back-to-back walls be connected to the same reinforcing element.

23.12.4.6 Placement of Second and Subsequent Facing Courses (Segmental Facings)

In case of segmental panel walls, facing panels should only be set at grade. A panel should not be placed on top of another panel, which is not completely backfilled.

Facing elements, which are out of alignment, should not be pulled back into place as this may damage the panels and reinforcements. Alignment correction can be done by removal of wall fill and reinforcing elements, followed by the resetting of the panels and then relaying the wall fill and reinforcement layer.

Vertical and horizontal alignment of each facing element should be checked prior to erection of the next panel level. Should any facing elements be out of alignment, the fill should be removed and the elements reset to proper tolerances.

23.12.4.7 Installation of Drainage

The drainage system is installed simultaneously with the placing and compaction of fill. The drainage fill should be constructed at the same rate as reinforced and retained fill, taking care to prevent any mixing with other soils. Perforated collection pipes are installed with the required slope. Figure 23.26 shows the view of finished reinforced earth wall.

23.12.5 Advantages of Reinforced Earth

The following are the main advantages of reinforced earth:

1. Systematic use of prefabricated elements permits easy and rapid construction.
2. Reinforced earth can be adapted to various slopes and soil conditions and requires only medium-size construction equipment.
3. The flexibility and light weight of the structure resulting from the use of reinforced earth allows construction even on relatively soft soils.
4. Reinforced earth structure is more economical than conventional reinforced concrete or masonry retaining walls.
5. The use of facing panels enable the designer to best fit the shape of the structure to the environment and to select an appropriate finish with suitable relief, texture, and color.

23.12.6 Applications

The most common application of reinforced earth is in the reinforced earth retaining structures (Figs. 23.26 and 23.27). It is also used in abutments of bridges, sloped walls and reinforced earth walls. Various applications of reinforced earth are illustrated in Fig. 23.28.

Figure 23.26 Finished view of a reinforced earth wall with differential grade.

Figure 23.27 Reinforced earth retaining wall for a highway in cutting.

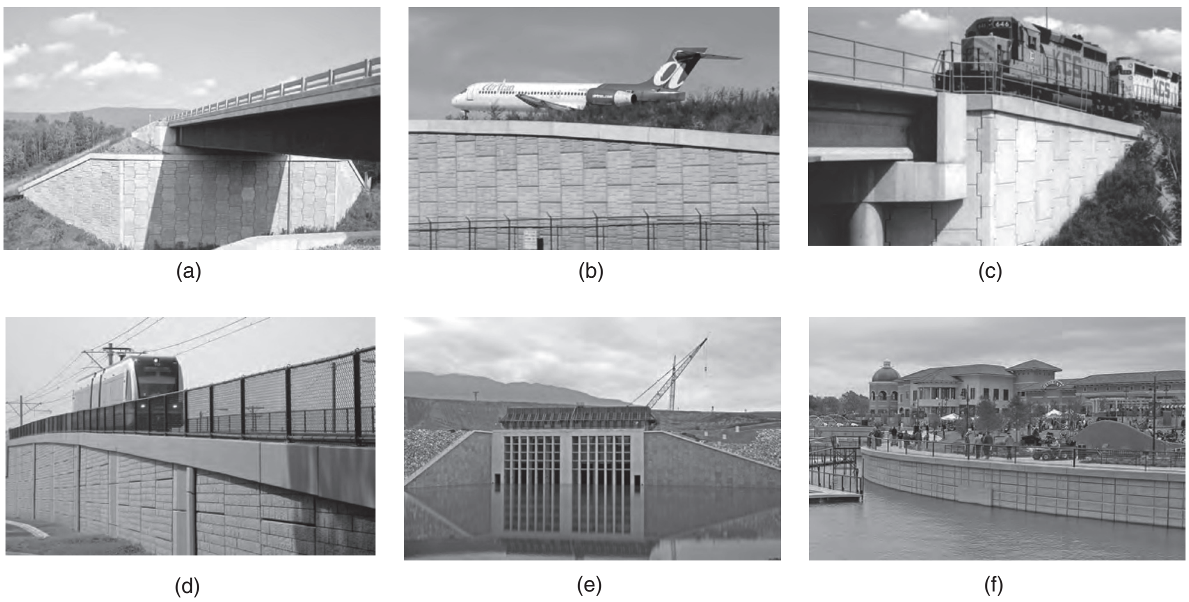

Figure 23.28 Applications of reinforced earth: (a) Bridges, (b) runways, (c) railway structures, (d) railway structures, (e) waterways and dams, and (f) waterways and dams. (From www.reinforcedearth.com)

23.13 Soil Nailing

Soil nailing is a technique where soil slopes, excavations, and retaining walls are passively reinforced by the insertion of relatively slender steel reinforcing bars.

23.13.1 Components of Soil Nailing

Figure 23.29 shows various components of a grouted soil nailing, which consist of the following:

1. **Reinforcing bars:** Solid or hollow steel reinforcing bars of minimum Fe415 grade are used as soil nails. These nails are placed in predrilled holes and grouted in place. For corrosion protection, all steel components are galvanized. If machine threading after galvanization is unavoidable, then proper zinc-based coating shall be applied onto the thread. Soil nails are usually installed at an inclination of 10°–20° below the horizontal and are primarily subjected to tensile stress due to deformation of the retained soil during subsequent excavation.
2. **Grout:** Grout is injected in the predrilled hole after placing the nail to fill the annular space between the nail and the surrounding soil. Grout transfers the stresses from the ground to the nail and also provides protection to the nail against corrosion. Either neat cement or sand–cement grout is used for this purpose. For conventional soil nail, the water/cement ratio of the grout mix ranges from 0.4 to 0.5. As most cementitious grout will experience some grout shrinkage, the resistance at the grout–soil interface of the nail will be significantly reduced when the grout shrinks. Non-shrink additive can be used to reduce breeding and grout shrinkage.
3. **Nail head:** The nail head is the threaded end of the soil nail and projects outside from the wall facing. It consists of a steel plate and nut in addition to the nail and is concreted into a square shape to fix the nail head rigidly to the facing. It ensures overall stability of the wall and transfers the bearing stresses from the soil to the nail. Typical nail head details are shown in Fig. 23.30.
4. **Hex nut, washer, and bearing plate:** Hex nut, washer, and bearing plate are attached to the nail head and are used to connect the soil nail to the facing.
5. **Centralizers:** PVC centralizers, as shown in Fig. 23.31, are used to ensure that the soil nails are centered in the drill hole.
6. **Temporary and permanent facing:** Nails are connected to the excavation and sloping surface by facing elements. Temporary facing is placed on the unsupported excavation prior to advancement of the excavation grades. It is provided in the form of gunite or shotcrete. Shotcrete or gunite can be a continuous flow of mortar or concrete mixes projected perpendicularly at high speed onto the exposed ground surface by means of pneumatic air blowing for dry mix or spraying for wet mix. It provides support to the exposed soil, helps in protection of the whole system, and acts as a bearing surface for the bearing plate. It also ensures accurate positioning of soil nails. Permanent facing is placed over the temporary facing after the installation of soil nails.

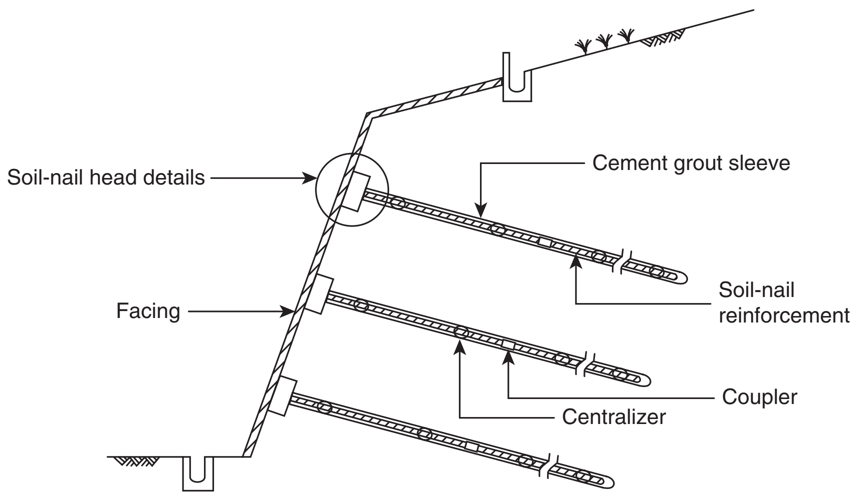

Figure 23.29 Components of grouted soil nailing.

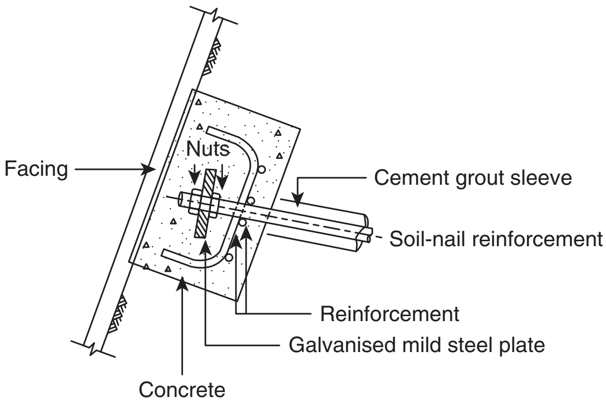

Figure 23.30 Nail head. (*Courtesy*: Geoguide7, 2008. With permission from Head of the Geotechnical Engineering Office and the Director of the Civil Engineering and Development, the Government of the Hong Kong Special Administrative Region.)

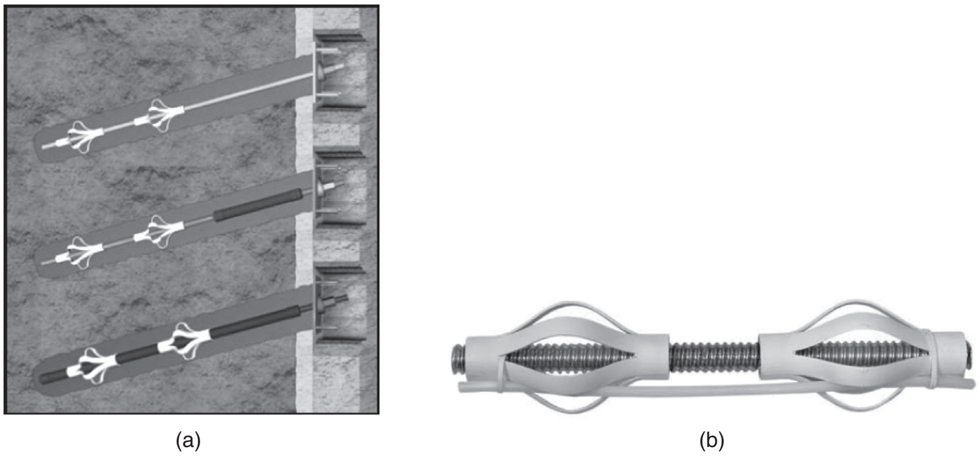

Figure 23.31 Grouted soil nail wall: (a) Cross section and (b) soil nail with centralizers. (After Prashant and Mukharjee, 2010)

7. **Drainage system:** Vertical geocomposite strip drains are used to ensure drainage of the soil nailing system for collection and transmission of seepage water that migrates toward the temporary facing. These are placed prior to the installation of the temporary facing.
8. **Corrosion protection:** Protective layers of corrugated synthetic material are used around soil nails to provide additional protection against corrosion. They are usually made of high-density polyethylene (HDPE) or PVC. A galvanized corrugated steel pipe gives better protection and is used where required. Typical corrosion protection details are shown in Fig. 23.32.

23.13.2 Types of Soil Nailing Systems

Depending on the technique used, the following are the different types of soil nailing systems:

1. **Driven nail:** In this method, nails are driven into the soil after excavation. This type of installation is very fast and economical. The stability of the system is derived from the direct frictional resistance between the nail and the soil. As the nails are in contact with the soil, they may get damaged in the long run due to corrosion due to soil/groundwater. As a solution, the nails may be given a protective coating without reducing the surface roughness.
2. **Grouted nail:** In this technique, holes are drilled into the soil after excavation and nails are placed in the predrilled holes. The hole is then filled with cement grout. The method takes more time for construction than the driven nail system. However, the nails are not in contact with the soil and hence are protected from corrosion due to soil/groundwater.

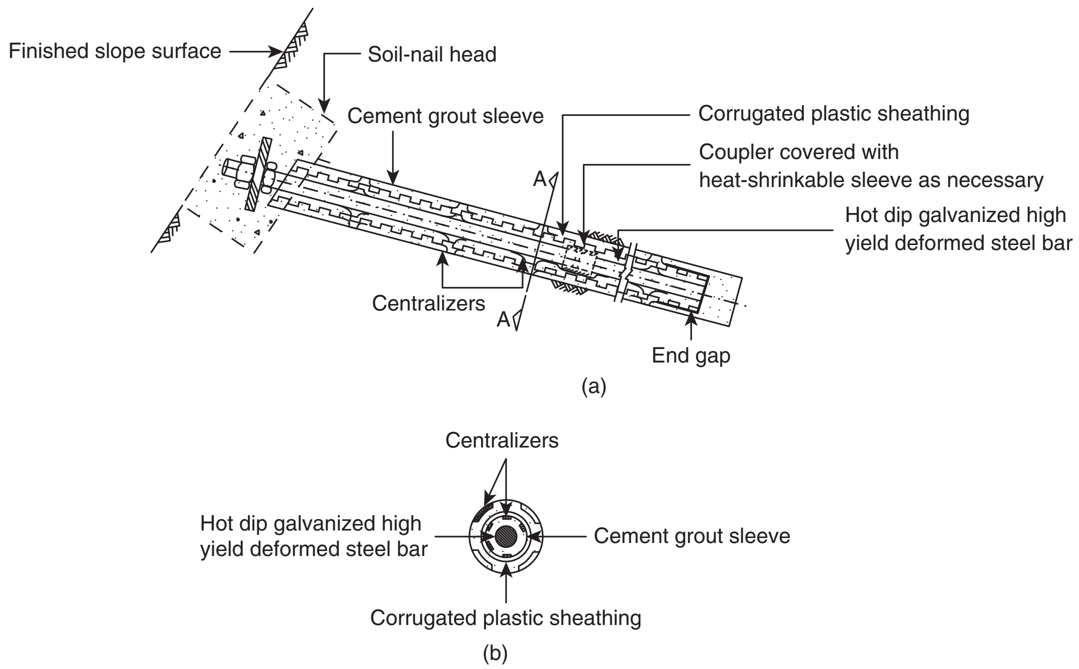

Figure 23.32 Typical details of corrosion protection: (a) Typical section and (b) section A – A. (*Courtesy:* Geoguide7, 2008. With permission from Head of the Geotechnical Engineering Office and the Director of the Civil Engineering and Development, the Government of the Hong Kong Special Administrative Region.)

3. **Self-drilling soil nail:** In this method, hollow bars are driven and grout is injected through the hollow bars simultaneously during drilling. This method is faster than grouted nailing and gives better corrosion protection than driven nailing.
4. **Jet-grouted soil nail:** It is an effective and economical method of constructing retaining wall for excavation support, support of hill cuts, bridge abutments, and highways.
5. **Launched soil sail**: Nails of 25 to 38 mm in diameter and up to 6 m or longer are fired directly into the soil with a compressed-air launcher. Used primarily for slope stabilization, this technique involves the least site disturbance.

23.13.3 Equipment Used for Soil Nailing

The following equipment are required for construction of soil nailing:

1. Drilling equipment.
2. Grout mixing equipment.
3. Shotcreting/Guniting equipment.
4. Compressor.

These equipment are discussed in the following subsections.

23.13.3.1 Drilling Equipment

There are few common types of drilling equipment, namely rotary air-flushed and water-flushed, down-the-hole hammer, tri-cone bit. It is important to procure the drilling equipment with sufficient power and rigid drill rods.

23.13.3.2 Grout Mixing Equipment

To produce uniform grout mix, a high-speed shear colloidal mixer should be considered. A powerful grout pump is essential for uninterrupted delivery of grout mix. If fine aggregate is used as filler for economy, a special grout pump shall be used.

23.13.3.3 Shotcreting/Guniting Equipment

The dry mix method will require a valve at the nozzle outlet to control the amount of water injecting into the high-pressurized flow of sand/cement mix. For controlling the thickness of the shotcrete, a measuring pin should be installed at fixed vertical and horizontal intervals to guide the nozzle man.

23.13.3.4 Compressor

The compressor should have the minimum capacity to deliver shotcrete at the minimum rate of 9 m^3/min. Sometimes, the noise of the compressor can be an issue if the work is in close proximity to a residential area, hospital, or school.

23.13.4 Construction of Soil Nailing systems

The soil nailing systems are constructed in the following steps:

1. **Excavate in lifts:** The excavation for soil nailing consists of making a 1.2- to 1.8-m vertical cut up to a depth 0.3–0.8 m below the elevation of the soil nails. The walls of excavation are stabilized and shotcreted the same day. Excavation for large projects is done using large earth-moving equipment. Small-size equipment are also used to provide the necessary face trimming prior to installation of the soil nail wall. After the lift has been excavated to the proper elevation, a level work area, or bench, 3–12 m wide, is constructed in front of the wall to accommodate the drilling equipment.
2. **Installation of nail:** Shotcrete is applied prior to installing the nails if there is a possibility of sloughing of the soil. Rotary and rotary/percussion drill rigs in general and augers in cohesive soils are used for the installation of the nail. Soil nails may also be driven into the ground mechanically. Soil nails are installed at a horizontal and vertical spacing of 1.2- to 1.8-m in a grid pattern, with the drill hole inclination being 10–20° below the horizontal. The holes are 100–200 mm in diameter and they may be cased or uncased, depending on the type of soil.
   Once the hole is drilled, the nail is installed and the hole is tremie-grouted with cement and water. The water/cement ratio is normally of the order of 0.45–0.50. The bar is centered in the drill hole by means of plastic centralizers spaced every 3 m along the length of the bar, as shown in Fig. 23.31. The length of the bar is based on the design, and may be about 0.6–1 times the overall cut height for the first level of nails.
3. **Placement of reinforcement and drainage:** Once the excavation is made and the soil nails are installed, reinforcing material, in the form of a welded wire mesh, is placed along the face of the excavation to reinforce the concrete facing. Alternately, reinforcing bars may be used for the length of the wall in lieu of the wire mesh. The reinforcing material may also be added to the ready-mix concrete at the plant, instead of placing the reinforcing prior to shotcreting. This is known as fiber-reinforced concrete. Some common types of reinforcing material are steel and synthetic fibers.
   Prefabricated geotextile strip drains, about 0.6 m wide, may be used to provide effective drainage behind the soil nail wall between the nails prior to applying the shotcrete facing. However, in most cases, drainage material is not installed for temporary soil nail walls.
4. **Shotcreting and installing bearing plates:** Shotcreting the facing is done after placing the reinforcement and drainage medium. For most temporary shotcrete walls, this is accomplished by construction of M-20 grade concrete layer of 75- to 100-mm thickness. The soil nail bearing plates are installed, immediately after shotcrete is applied, on the fresh shotcrete facing and their nuts are hand-tightened.
5. **Completion to full height:** After the shotcrete has been installed and the soil nails have cured for 3 days, the process is repeated in lifts until the predetermined subgrade elevation is reached. Once the temporary shotcrete wall is constructed, the permanent facing is provided by additional layer of 100–150-mm-thick shotcrete with required reinforcement over the existing 75–100 mm (3–4 in.) thick shotcrete. In lieu of using shotcrete as the permanent facing, there are several other choices. A concrete wall or a variety of concrete block or precast panel systems may also be used as an alternative to permanent shotcrete.
6. **Corrosion protection:** For temporary systems, single-corrosion protection for the soil nails is normally adequate. The nail is simply a bare bar, with sufficient grout cover achieved by providing centralization of the nail in the drill hole. For permanent soil nail systems, the nail should be provided with an epoxy or galvanized coating in non-aggressive soils. In more aggressive soils, fully encapsulated nails are recommended.

23.13.5 Stability Analysis of Soil Nailing System

The stability analysis of the soil nailing system can be done through the usual methods, considering the effect of the pull-out capacity of the soil nails.

23.13.5.1 Stability Analysis with Plane Failure Surface

Following are the several ways in which a soil nailing may undergo failure:

1. **External failure modes:**
   - Rotation failure.
   - Sliding failure.
   - Bearing failure.
2. **Internal failure modes:**
   - Failure of ground around soil nails.
   - Soil nail head bearing failure.
   - Local failure between soil nails.
   - Tension failure of soil nails.
   - Pull-out failure at grout–soil interface.
   - Bending or shear failure of soil nails.
   - Connection failure of soil nail head.
   - Connection failure of facing.

A typical soil nailing with a plane failure surface is shown in Fig. 23.33. The factor of safety is given by

$$F = \frac{c_1L_1 + W_1\cos\theta\tan\phi_1 + c_2L_2 + W_2\cos\theta\tan\phi_2 + \sum_{i=1}^{n} T_{\text{cm}i}\cos(\theta-\lambda)}{\left[(W_1+W_2)\sin\theta - \sum_{i=1}^{n} T_{\text{cm}i}\sin(\theta-\lambda)\right]} \tag{23.10}$$

where L is the length of the failure surface, W the weight of the soil wedge enclosed by the failure surface and the facing, $T_{\text{cm}i}$ the pull-out capacity of soil nails per unit spacing given by Eq. (23.11) or (23.12), n the number of soil nails, and θ the angle of the failure surface with the horizontal.

$$T_{\text{cm}} = \frac{q_{\text{u}}pl_{\text{c}}}{S} \tag{23.11}$$

where q_{u} is the limiting bond stress or bond strength of the soil–nail interface (obtained from a pull-out test in the field), p the perimeter of the soil nail, and l_{c} the pull-out length (which is the length of the nail behind the failure surface).

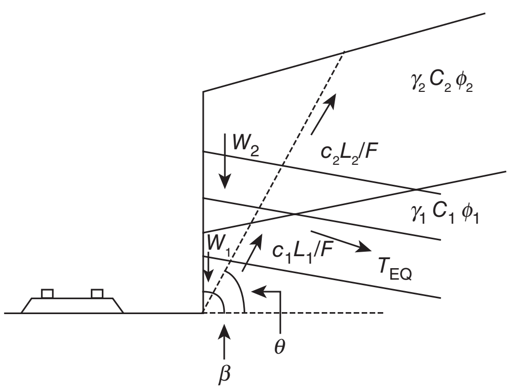

Figure 23.33 Stability analysis for soil nailing with plane failure surface. (After Prashant and Mukherjee, 2010)

In the absence of field test data, the pull-out capacity may be obtained from

$$T_{cm} = \frac{\left(c + \sigma'_v \tan\phi_m\right) p l_c}{S} \tag{23.12}$$

$$\phi_m = \frac{2}{3}\phi \tag{23.13}$$

where ϕ_m is the mobilized friction angle of the soil–nail interface, ϕ the effective angle of internal friction of the soil, c the unit cohesion of the soil, σ'_v the effective vertical stress at the mid-length of a soil nail (including surcharge acting, if any), and S the horizontal spacing of soil nails.

23.13.5.2 Bishop's Simplified Method of Slices

Figure 23.34 shows a typical soil nailing with a circular failure surface. Bishop's simplified method of slices can be applied for stability analysis and the procedure is similar to the one discussed in Chapter 17. The factor of safety is determined from

$$F = \frac{\sum_{i=1}^{n}(cL + N\tan\phi)}{\sum_{i=1}^{n}[W\sin\alpha - T_n\cos(\alpha+\lambda)]} \tag{23.14}$$

where T_n is the mobilized nail tension given by Eq. (23.15) or (23.16) and λ the slope angle of nails below the horizontal.

All other parameters are as defined in Bishop's simplified method of slices given in Chapter 17.

$$T_n = \frac{q_u p l_c}{F_t} = \frac{T_{cm} S}{F_t} \tag{23.15}$$

where q_u is the limiting bond stress or bond strength of the soil–nail interface (obtained from a pull-out test in the field), p the perimeter of the soil nail, l_c the pull-out length (which is the length of the nail behind the failure surface), and F_t the factor of safety to take care of variability in measurement = 1.5.

In the absence of field test data, the mobilized nail tension may be obtained from

$$T_n = \frac{(c + \sigma'_v \tan\phi_m) p l_c}{F_t} = \frac{T_{cm} S}{F_t} \tag{23.16}$$

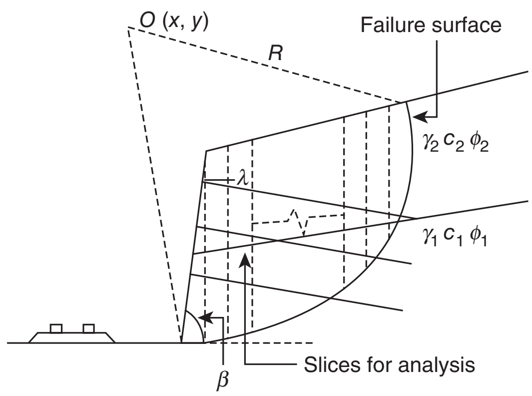

Figure 23.34 Bishop's simplified stability analysis for soil nailing. (After Prashant and Mukharjee, 2010)

$$\phi_m = \frac{2}{3}\phi \tag{23.17}$$

where ϕ_m is the mobilized friction angle of the soil–nail interface, ϕ the effective angle of internal friction of the soil, c the unit cohesion of the soil, S the horizontal spacing of soil nails, and σ'_v the effective vertical stress at the mid-length of a soil nail (including surcharge acting, if any).

23.13.6 Applications

Soil nail walls can be considered as retaining structures for any permanent or temporary vertical or near-vertical cut construction. The relatively wide range of available facing systems allows for various esthetic requirements to be addressed. The following are the various applications of soil nailing:

1. Soil nailing is an effective and economical method of constructing retaining wall for excavation support, support of hill cuts, bridge abutments, and highways.
2. Soil nailing is typically used to stabilize existing slopes or excavations where top-to-bottom construction is advantageous. Soil nail walls are particularly well suited to excavation applications for ground conditions that require vertical or near-vertical cuts.
3. Soil nails have been used successfully in highway and railway cuts, and for end-slope removal under existing bridge abutments during underpass widening; soil nailing is attractive because it tends to minimize excavation, provides reasonable right-of-way and clearing limits, and hence minimizes environmental impacts within the transportation corridor. In these applications, soil nailing is a viable and economical option for supporting vertical cuts, particularly in locations where site constraints are more predominant and project duration is very limited.
4. Soil nails have also been used for the repair, stabilization, and reconstruction of existing retaining structures and tunnel portals.

Figures 23.35 and 23.36 show examples of the use of soil nail walls in temporary and permanent cut applications. Figures 23.37–23.39 illustrate the application of soil nailing for highway and railway cuts.

23.13.7 Advantages of Soil Nailing

There are several advantages of soil nailing over other ground anchoring techniques:

1. Soil nailing causes less disruption to traffic and less environmental impact.
2. Installation of soil nail walls is relatively simpler and faster, and uses less construction materials.

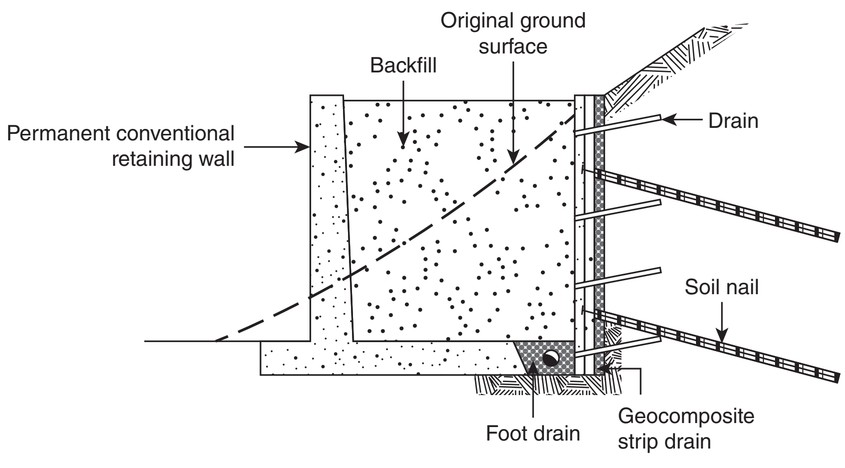

Figure 23.35 Soil nailing for temporary shoring.

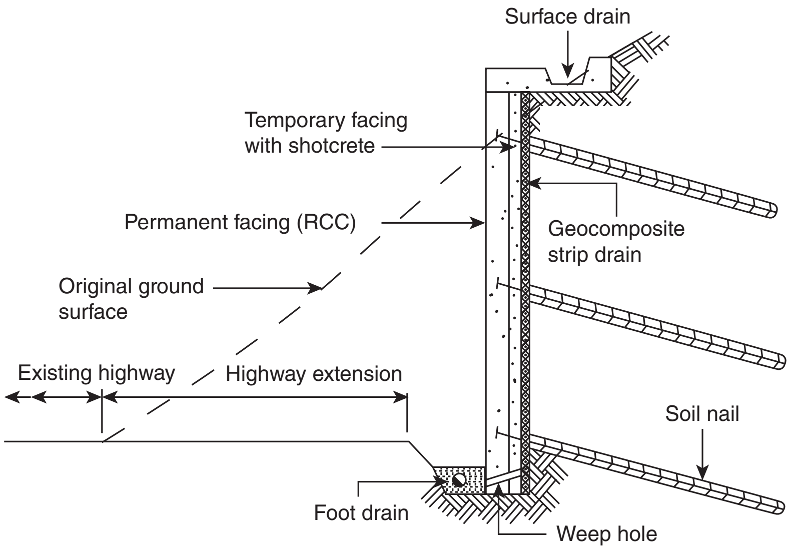

Figure 23.36 Permanent soil nailing for road widening.

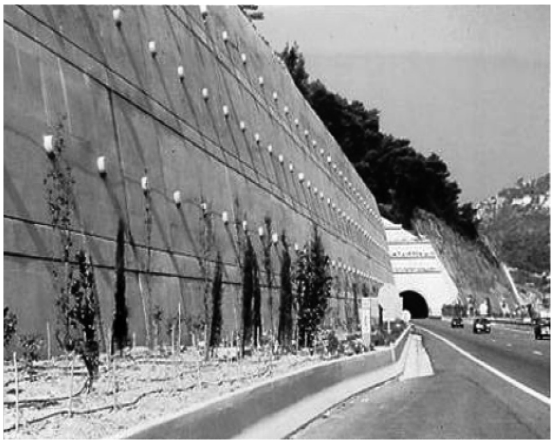

Figure 23.37 Soil nailing for highway. (After Prashant and Mukharjee, 2010)

Figure 23.38 Soil nailing for railway. (After Prashant and Mukharjee, 2010)

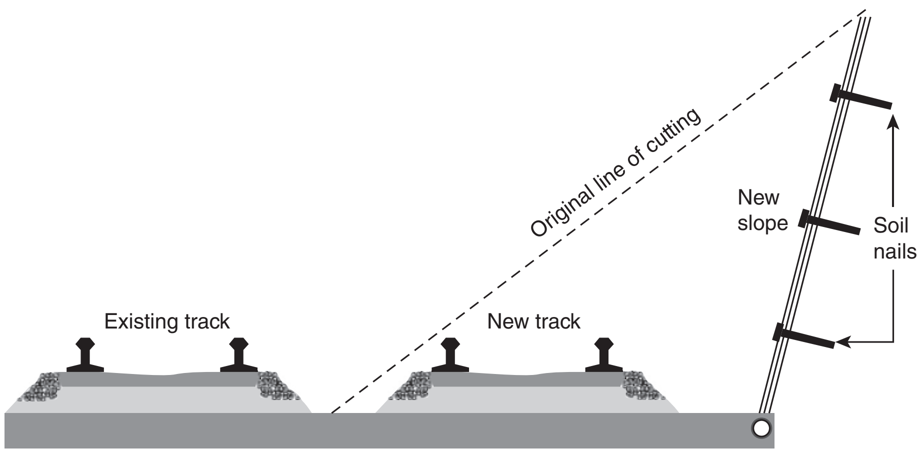

Figure 23.39 Soil nailing for additional railway track. (After Prashant and Mukharjee, 2010)

3. It can be advantageously used in sites with remote access because only simpler and smaller equipment is needed for construction.
4. Easy adjustments can be made in nail positions when obstructions, such as stones, cobbles, or underground utilities, are encountered during drilling without additional cost or time.
5. The use of soldier beams is not required unlike in ground anchors and hence overhead construction requirements are small in soil nailing.
6. As the number of reinforcing elements is more in soil nailing than in ground anchors, changes in design can be easily made without affecting the safety.
7. When used in excavations, soil nailing does not cause congestion of spacing as in bracing systems.
8. Soil nailing showed better performance during seismic events due to overall flexibility of the system.
9. Soil nail walls are more economical than conventional concrete retaining walls.

23.13.8 Disadvantages of Soil Nailing

Some of the disadvantages of soil nailing system are as follows:

1. Certain minimum deformation is required in soil nailing to mobilize resistance. Hence, soil nailing is not recommended in applications where strict deformation control is a requirement. Post-tensioning of the nails can be done to overcome this limitation, but this causes an increase in the project cost.
2. Underground utilities will be damaged if proper care is not taken during positioning and drilling for the soil nails.

23.14 Geosynthetics

Geosynthetics are a rapidly emerging family of polymeric materials used in geotechnical engineering in a wide variety of applications. It should be noted that geosynthetics are originally developed and are known as geotextiles. They also include other types of geotextiles such as geomembranes, geogrids, geocomposites, etc. The word *geosynthetic* is now replacing the family name of geotextiles, but in many places, including in this text, both geosynthetics and geotextiles are used synonymously.

23.14.1 Types of Geosynthetics

The following are the major types of geosynthetics:

1. Geotextiles.
2. Geomembranes.
3. Geogrid.
4. Geonets.
5. Geocomposites.

These are explained as follows:

1. **Geotextiles:** Geotextiles are porous flexible polymeric fabrics used to serve the functions of separation, reinforcement, filtration, and/or drainage. They are made of polypropylene, polyster, or, occasionally of, polyethylene or polyamide. The term “geotextile” was coined by J.P. Giroud and includes geowebs, geomats, geonets, geogrids, etc. Geotextiles can be broadly defined as “planar, permeable, polymeric synthetic textiles which may be woven, non-woven or knitted used in contact with soil and/or any other geotechnical material in civil engineering applications.” Typical specimens of geotextiles are shown in Fig. 23.40.
2. **Geomembranes:** Geomembranes, shown in Fig. 23.41, are continuous membrane-type liners and barriers composed of asphaltic, polymeric materials or their combination with sufficiently low permeability so as to control fluid migration across them in a geotechnical engineering–related project. Geomembranes are geotextiles that are essentially solid sheets of plastic that are available in various thicknesses. Primary uses for geomembranes are as moisture barriers for earth dams and cutoff walls, as pond liners, and as a containment medium at the base of salt storage bins.

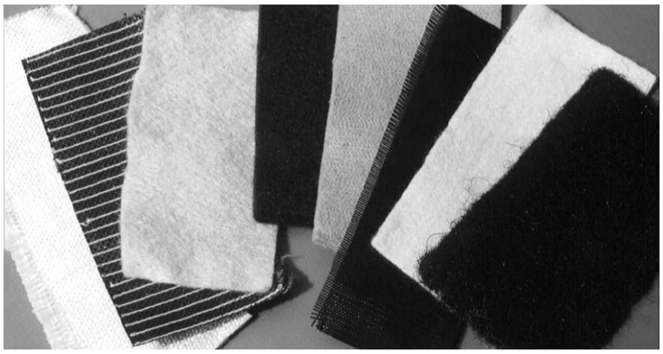

Figure 23.40 Geotextiles.

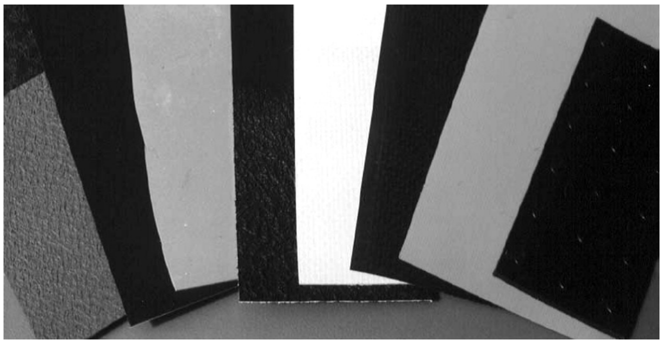

Figure 23.41 Geomembranes.

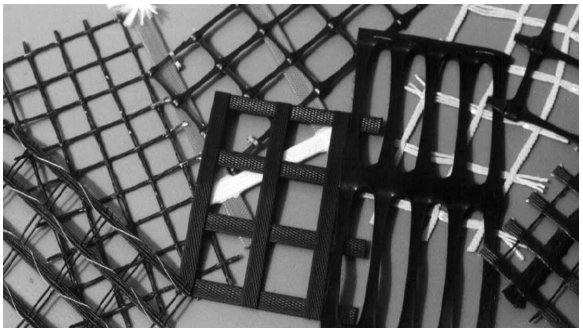

Figure 23.42 Geogrids.

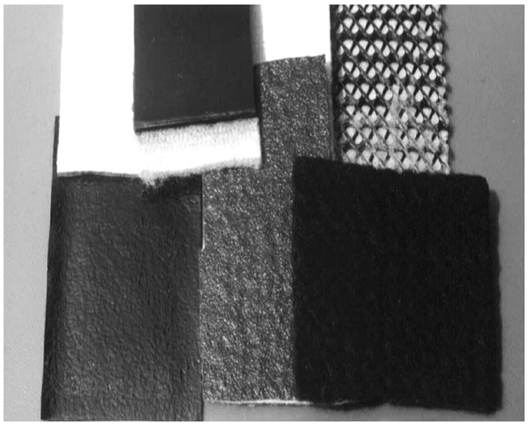

Figure 23.43 Geocomposites.

3. **Geogrid:** Geogrid is a synthetic planar structure formed by a regular network of tensile elements with apertures of sufficient size to allow interlocking with the surrounding soil, rock, or any other geotechnical material. Geogrids are a form of geotextile that have high-dimensional stability and high-tensile modulus with very low elongation. They have been used mostly in stabilizing earth slopes, embankments placed over pipe culvert trenches, and embankments placed on marginal foundation soils. They are beginning to be used to increase the strength of the aggregate base layer in pavement structures. Typical specimens of geogrids are shown in Fig. 23.42.
4. **Geocomposites:** Geocomposites are obtained by combining geogrids and geotextiles or geomembranes with woven or non-woven geotextiles or geogrids for specific applications such as drainage, erosion control, bank protection, etc. Typical specimens of geocomposites are shown in Fig. 23.43.

5. **Geonets:** Geonets are deformed or non-deformed net-like polymeric materials used for in-plane drainage. They are obtained by partial melting of strips, rigid filaments, or extracted strands. They are also called geowebs or geomats.

23.14.2 Joining Panels

Two geotextile sections can be joined by sewing, stapling, heat welding, tying, and gluing. If the primary purpose is to hold the material in place during installation, simple overlapping and staking or nailing to the underlying soil may be adequate. However, where two joined sections must withstand tensile stress, sewing is the most reliable joining method.

23.14.3 Materials

Geosynthetics are manufactured from polypropylene, polyester, polyethylene, polyamide (nylon), and polyvinylidene chloride. They are made from synthetic long-chain or continuous polymeric filaments or yarns, having a composition of at least 95%, by weight, of these polymers. Polypropylene and polyester are the most used. Steel wires, glass wires, and naturally biodegradable fibers are also used as geosynthetics. Geotextiles can be broadly categorized as (a) woven geotextiles and (b) non-woven geotextiles.

23.14.3.1 Woven Geotextiles

Woven geotextiles are manufactured from monofilament yarns that are woven into a uniform pattern with distinct and measurable openings so that the yarns will retain their relative position with regard to each other. The yarns contain stabilizers and/or inhibitors to enhance their resistance to ultraviolet (UV) light or heat exposure. This type of woven geotextile is relatively inexpensive and is used for separation, that is, the prevention of intermixing of two materials such as aggregate and fine-grained soil. Multi-filament woven construction produces the highest strength and modulus of all the constructions but is also the highest in cost. The manufacturing process can be varied so that the finished geotextile has equal or different strengths in the warp and fill directions. Woven construction produces geotextiles with a simple pore structure and narrow range of pore sizes or openings between fibers.

The woven textiles are most useful in applications where it is necessary to separate materials (such as an aggregate base material from a marginal strength clay) or to stabilize an embankment or earth slope. The woven fabrics give moderate-to–high tensile strength with low elongation. Typical specimens of woven geotextiles are shown in Fig. 23.44.

23.14.3.2 Non-Woven Geotextiles

Non-woven geotextiles are manufactured from randomly oriented fibers that have been mechanically bonded together by the needle-punched process. The filaments contain stabilizers and/or inhibitors to enhance their resistance to UV light or heat exposure. Non-woven geotextiles are generally thicker than woven geotextiles. Non-woven geotextiles may be made from either continuous filaments or staple fibers. The fibers may be given oriented random or preferred alignments within the plane of the geotextile. In the spun-bonding process, filaments are extruded and laid directly on a moving belt to form the mat, which is then bonded by needle punching, heat resin, or combination bonding. Typical specimens of non-woven geotextiles are shown in Fig. 23.45.

Non-woven textiles are most useful in drainage applications due to the size of the openings in the fabric and the way they are constructed. They are generally used with an indented plastic core to make composite drains and as a trench liner in underdrains and trench drains. The tensile strength of non-woven fabrics is low to moderate, and they should be used with caution, because of the high level of elongation that will take place in reaching their peak strength. These types of geotextiles are used primarily for the following:

1. Subsurface drainage.
2. Roadway separation.
3. Railroad stabilization.
4. Hard armor underlayment.
5. Landfill leachate collection.
6. Underground detention/retention systems.

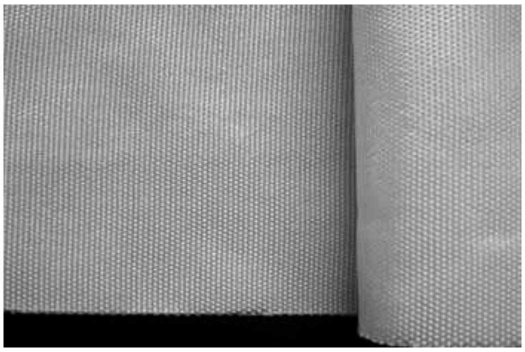

Figure 23.44 Woven geotextile.

Figure 23.45 Non-woven geotextile.

23.14.3.3 Geotextile Durability

Exposure to sunlight degrades the physical properties of polymers. The rate of degradation is reduced by the addition of carbon black but not eliminated.

23.14.4 Functions of Geosynthetics

Geosynthetics or geotextiles perform one or more basic functions such as: filtration, drainage, separation, erosion control, sediment control, reinforcement, and also as moisture barrier. In any one application, a geosynthetic or geotextile may be performing several of these functions.

1. **Filtration:** A geotextile acts as a filter when it allows liquid to pass normal to its plane while preventing soil particles from being carried away by the liquid current. The earliest and most widely known use of geotextiles is in filter applications. Filtration is also the most used function of geotextiles. Geotextiles can be effectively used as filters in underground drains or behind retaining walls. In this application, the geotextile is placed in contact with soil to be drained at required gradient. The plane of the geotextile is kept normal to the expected direction of water flow. The capacity for flow of water normal to the plane of the geotextile is referred to as permittivity. Permittivity and pore (openings in geotextile) size are the two important characteristics for filtration. Non-woven fabrics are commonly used for filtration because of their high flow capacity and small pore size. A geotextile-lined drainage trench along the edge of a road pavement is an example using a geotextile as a filter. Long-term clogging should be checked when geotextiles are used for filtration.
2. **Drainage:** When functioning as a drain, a geotextile acts as a conduit for the movement of liquids or gases in the plane of the geotextile. This function is frequently called fluid transmission. Examples are geotextiles used as wick drains and blanket drains. The relatively thick non-woven geotextiles are most commonly used for drainage. Selection should be based on transmissivity, which is the capacity for in-plane flow. They are known to be effective in short-duration applications. Long-term clogging of geotextile drains should be checked.
3. **Erosion control:** When geotextile protects soil surfaces from the tractive forces of moving water or wind and rainfall erosion, the function is called as erosion control. Geotextiles can be used in ditch linings to protect erodible fine sands or cohesionless silts. Geotextiles are also used for temporary protection against erosion on newly seeded slopes. After the slope has been seeded, the geotextile is anchored to the slope holding the soil and seed in place until the seeds germinate and vegetative cover is established. The erosion control function can be thought of as a special case of the combination of the filtration and separation functions.
4. **Sediment control:** A geotextile serves to control sediment when it stops particles suspended in surface fluid flow, while allowing the fluid to pass through. After some period of time, particles accumulate against the geotextile, reducing the flow of fluid and increasing the pressure against the geotextile. Examples of this application are silt fences placed to reduce the amount of sediment carried off construction sites and into nearby water courses. The sediment control function is actually a filtration function.

5. **Reinforcement:** In the most common reinforcement application, the geotextile interacts with soil through frictional or adhesion forces to resist tensile or shear forces. To provide reinforcement, a geotextile must have sufficient strength and embedment length to resist the tensile forces generated, and the strength must be developed at sufficiently small strains (i.e., high modulus) to prevent excessive movement of the reinforced structure. To reinforce embankments and retaining structures, a woven geotextile is recommended because it can provide high strength at small strains.
6. **Separation:** Separation is the process of preventing two dissimilar materials from mixing. In this function, a geotextile is most often required to prevent the undesirable mixing of fill and natural soils or two different types of fills. A geotextile is placed between a railroad subgrade and track ballast to prevent contamination and resulting strength loss of the ballast by intrusion of the subgrade soil. In construction of roads over soft soil, a geotextile is placed over the soft subgrade and then gravel or crushed stone placed on the geotextile. The geotextile prevents mixing of the two materials. This enhances the drainage of the base course for its life time. When the subgrade consists of fine-grained soil, the use of geotextile as separator enhances the life of the pavement considerably.
7. **Moisture barrier:** Both woven and non-woven geotextiles can serve as moisture barriers when impregnated with bituminous, rubber–bitumen, or polymeric mixtures. Such impregnation reduces both the cross-plane and in-plane flow capacity of geotextiles to a minimum. This function plays an important role in the use of geotextiles in paving overlay systems. In such systems, the impregnated material seals the existing pavement and reduces the amount of surface water entering the base and the subgrade. This prevents a reduction in strength of these components and improves the performance and life of the pavement system.

The functions of geotextiles in various applications are shown in Table 23.3.

23.14.5 Properties of Geosynthetics

A list of important properties of geosynthetics for different functions is furnished as follows:

1. **Basic physical properties:**
   - Mass per unit area.
   - Thickness.
   - Roll width.
2. **Mechanical properties:**
   - Tensile strength.
   - Tensile modulus.
   - Seam strength.
   - Interface friction.
   - Fatigue resistance.
   - Creep resistance.

Table 23.3 Functions of geotextiles

S. No.	*Application*	*Functions of Geotextiles*			
		Separation	*Filtration*	*In-Plane Drainage*	*Reinforcement*
1.	Roads, railways, and subgrade stabilization	P	S	…	S
2.	Drainage	S	P	…	…
3.	Embankments	S	P	P	S
4.	Coastal and river protection	P	P	…	S
5.	Land reclamation	S	P	…	S
6.	Asphalt reinforcement	…	…	…	P
7.	Earth reinforcement	…	…	…	P

P, Primary function; S, Secondary function.

3. **Hydraulic properties:**
   - Compressibility.
   - Opening size.
   - Permittivity.
   - Transmissivity.
4. **Constructability/Survivability:**
   - Burst resistance.
   - Puncture resistance.
   - Penetration resistance.
   - Tear strength.
5. **Durability:**
   - Abrasion resistance.
   - UV stability.
   - Temperature stability.
   - Chemical stability.
   - Biological stability.
   - Wetting and drying stability.

The following types of tests are required to be carried out for geosynthetics:

1. Soil tests.
2. In-isolation tests.
3. In-soil tests.
4. Prototype tests.

23.14.6 Applications of Geosynthetics

Important applications of geosynthetics are as follows:

1. Geotextile-reinforced retaining walls.
2. Reinforced embankments over soft soils.
3. Subgrade stabilization.
4. Subsurface drainage.
5. Asphalt overlay.
6. Erosion control.
7. Geomembrane protection.
8. Miscellaneous Applications

Some of the applications of geotextiles have been discussed as follows.

23.14.6.1 Geosynthetics-Reinforced Soil Walls

Geosynthetics-reinforced soil walls are an economical and technically more viable alternative to conventional RCC retaining walls. In this type of construction, the earth is filled in layers separated by geotextiles or geogrids. The geotextile acts as reinforcement and resists the horizontal stresses due to lateral displacement of the soil. The wall face may be vertical or inclined based on structural reasons (internal stability), ease of construction, or architectural purposes. All geotextiles are equally spaced so that construction is simplified.

Geotextiles exposed to UV light may degrade quite rapidly. Hence, a protective coating should be applied to the exposed face of the wall at the end of construction. An application of emulsified asphalt at about 1 L/m^2 or spraying with a low viscosity water–cement mixture is recommended. To protect the face of the wall from vandalism, a 7.5-cm layer of gunite can be applied. This can be done by projecting concrete over a reinforcing mesh manufactured from No. 12 wires (about 2.0-mm diameter), spaced 5 cm in each direction, supported by No. 3 (about 6.0-mm diameter) reinforcement bars inserted between geotextile layers to a depth of 0.9 m.

To ensure the fast removal of seeping water in a permanent structure, it is recommended to replace 30–60 cm of the natural foundation soil (in case it is not free draining) with a crushed-stone foundation layer to facilitate drainage from within and behind the wall. The crushed rock may be separated from the natural soil by a heavyweight geotextile.

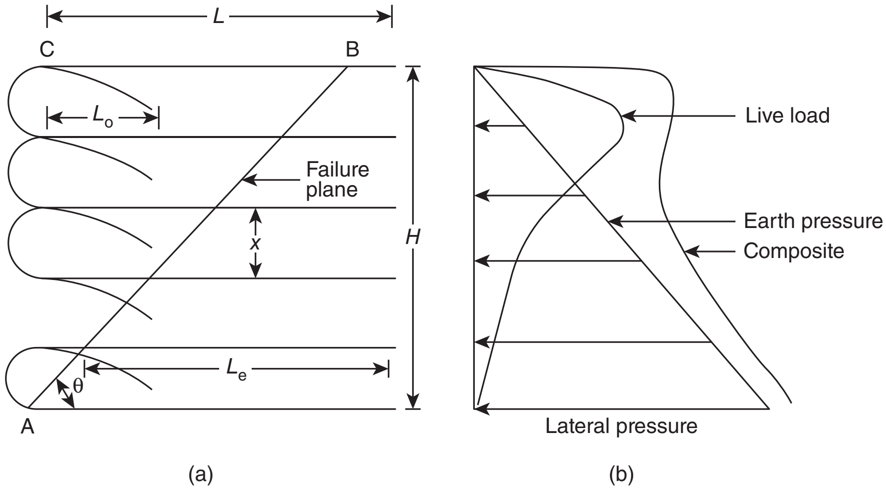

Figure 23.46 Geotextile-reinforced soil wall: (a) Backfill and (b) lateral pressure.

1. **Materials:** The soil wrapped by the geotextile sheets is termed "retained soil." This soil must be free draining and non-plastic. The ranking (most desirable to less desirable) of various retained soils for permanent walls using the Unified Soil Classification System is as follows: SW, SP, GW, GP, and GM or SM. The amount of fines in the soil is limited to 12%. Gravel, especially if it contains angular grains, can puncture geotextile sheets during construction. Consequently, consideration must be given to geotextile selection so as to resist possible damage. If a geotextile possessing high puncture resistance is available, then GW and GP should replace SW and SP, respectively, in their ranking order. A minimum of 95% of the standard Proctor maximum dry density should be attained during construction.
2. **Backfill soil:** The soil supported by the reinforced wall (the soil to the right of L in Fig. 23.46a) is termed "backfill soil." Generally, backfill specifications used for conventional retaining walls should be employed here as well. Clay, silt, or any other material with low permeability should be avoided next to a permanent wall.
3. **Calculation of Earth pressure:** Lateral earth pressure at any depth below the top of the wall [Fig. 23.46(b)] is given by

$$p_o = K_o \gamma h \tag{23.18}$$

where p_o is the lateral earth pressure on the wall at depth h below the top of the wall, the coefficient of earth pressure at rest, $K_o = (1 - \sin\phi)$, and γ the unit weight of soil.
The failure surface makes an angle θ, given by

$$\theta = 45 + \frac{\phi}{2} \tag{23.19}$$

The pressure due to a live load is computed using Eq. (23.20), assuming that the live load acts as a point load at the surface

$$p_1 = \frac{Px^2h}{R^5} \tag{23.20}$$

where p_1 is the pressure due to the live load, P the magnitude of the point load, x the horizontal normal distance between the load and the wall, h the vertical distance of the point where stress is calculated from the position of the point load and,

$$R = \sqrt{x^2 + y^2 + z^2} \tag{23.21}$$

where y is the horizontal distance parallel to the wall between the load and the point where the pressure is calculated.

4. **Calculation of pullout resistance:** A sufficient length of geotextile must be embedded behind the failure plane to resist pullout. Thus, in Fig. 23.46(a), only the length, L_e, of the fabric behind the failure plane AB would be used to resist pullout. Pullout resistance can be calculated from

$$P_A = 2\gamma d L_e \tan\left(\frac{2}{3}\phi\right) \tag{23.22}$$

where P_A is the pullout resistance, d the depth of retained soil below the top of the retaining wall, γ the unit weight of retained soil, ϕ the angle of internal friction of retained soil, and L_e the length of embedment behind the failure plane. The length of geotextile required is given by

$$L_e = \frac{P_A F}{2\gamma d \tan[(2/3)\phi]} \tag{23.23}$$

where P_A is the fabric tensile strength and F the factor of safety = 1.5–1.75. The minimum length of the fabric required is 0.9 m.

The length of fabric overlap for the folded portion of the fabric at the face is computed from

$$L_o = \frac{p_o X F}{2\gamma d_F \tan[(2/3)\phi]} \tag{23.24}$$

The minimum length of overlap should be 0.9 m to ensure adequate contact between layers.

5. **Construction:** Geotextiles have been used for the construction of reinforced earth retaining walls, which can provide both facing elements and stability simultaneously. The process of construction of the wall with granular backfill is shown in Fig. 23.47 and consists of the following steps:
   - Level the working surface.
   - Lay the geotextile sheet of required width on the surface, with 1.5- to 2-m length of the geotextile draped over a temporary wooden form.
   - Backfill over this sheet with granular soil and compact it using a roller of suitable weight, as shown in Fig. 23.47(a).
   - After compaction, fold the geotextile sheet as shown in Fig. 23.47(a). Lay down the second sheet and continue the process as before as shown in Figs. 23.47(b) and (c). The completed reinforced earth wall is shown in Fig. 23.47(d).

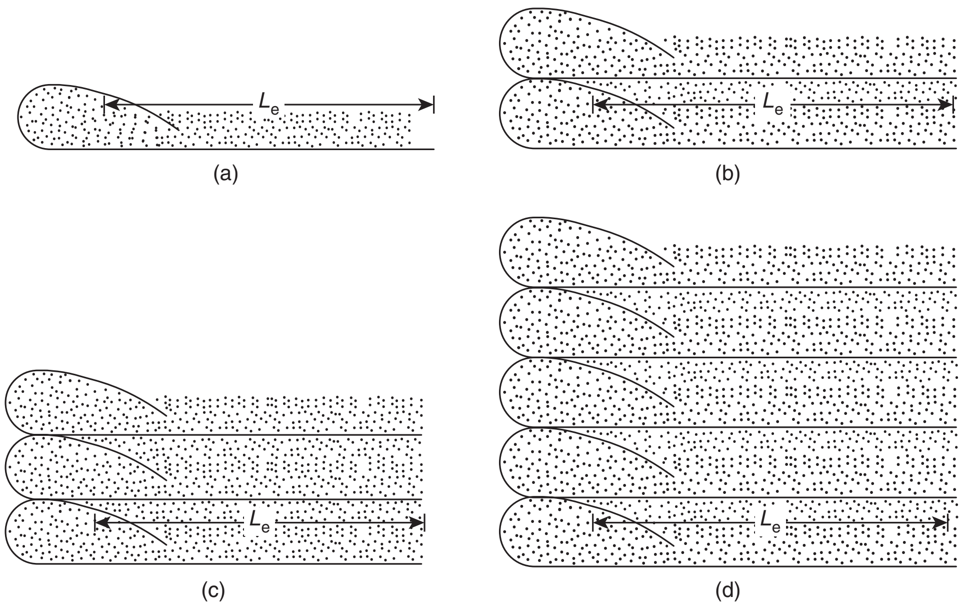

Figure 23.47 Construction of geotextile-reinforced retaining wall: (a) Laying first sheet, (b) laying second sheet, (c) laying third sheet, and (d) completed wall (without facing elements).

The front face of the wall can be protected by shotcrete or gunite or using prefabricated concrete panels. The design of geotextile-reinforced earth walls is similar in principle to that of reinforced earth walls.

The use of geotextiles allows a significant reduction in the amount of concrete required and decreases the cost and time of wall construction.

6. **Advantages:** Thus, the advantages of reinforced soil walls include the following:
   - The quantity of concrete required will be considerably less than for RCC retaining walls, saving in cost.
   - Concrete is required only for facing elements meant for esthetics, and the facing elements are subjected to marginal stresses only and their weight is also less. Hence, heavy foundations are not required as for RCC retaining walls.
   - The wall can be constructed even in weak soils where RCC retaining wall cannot be used or may require deeper foundation/some treatment.
   - Construction usually is simple, easy, and rapid. It does not require skilled labor or specialized equipment. Many of the components are prefabricated, allowing relatively quick construction.
   - There is no problem of corrosion of reinforcement as in the cased of reinforced earth retaining walls.
   - They are relatively flexible and can tolerate large lateral deformations and large differential vertical settlements.
   - Because of their flexibility, these walls can be constructed in areas where poor foundation material exists or in areas susceptible to earthquake activity.
   - As the facing is not a structural component, prefabricated panels of a wide variety of materials, size, shape, color, and architectural finish can be used, considerably improving the aesthetics.
7. **Disadvantages:** Some disadvantages of geotextile-reinforced walls over conventional concrete walls are the following:
   - Some decrease in geotextile strength may occur because of possible damage during construction.
   - Some decrease in geotextile strength may occur with time at constant load and soil temperature.
   - The construction of geotextile-reinforced walls in cut regions requires a wider excavation than conventional retaining walls.
   - Excavation behind the geotextile-reinforced wall is restricted.
   - Since geotextile application to walls is relatively new, long-term effects such as creep, aging, and durability are not known based on actual experience. Therefore, a short life, serious consequences of failure, or high repair or replacement costs could offset a lower initial cost.

23.14.6.2 Reinforced Embankments Over Soft Soils

Embankments constructed over soft soils undergo large settlements and prolonged period of construction to avoid shear failure of the foundation soil. The foundation soil underlying the embankment is too weak to permit the construction of the embankment to the required height.

The use of geotextiles and geogrids facilitates rapid and economical construction of embankments over such soils. A geotextile/geogrid is used at the interface between the soft soil and the embankment. The geotextile serves reinforcement, separation as well as filtration functions, enabling rapid construction of the embankment to the required height.

From the conventional soil mechanics analysis, a design can be generated that provides the required reinforcement strength of the geosynthetic. The construction is easy, simple, and rapid.

The geosynthetic is placed over the foundation soil, with minimal disturbance to the existing materials. The embankment is built using conventional construction equipment until the required height is reached.

The geosynthetic is placed over the surface of the soft soil and the embankment is constructed over the geosynthetic using the conventional procedure, as shown in Fig. 23.48. One or more layers of the geosynthetic may be used to provide the reinforcement necessary for embankment stability. A granular layer may be used between the embankment and soft soil for effective drainage of pore water. The geotextiles used between the soft soil, granular layer, and the embankment prevent inter-mixing of these materials, thus providing the separation function.

23.14.6.3 Subgrade Stabilization

A geosynthetic can improve the load-carrying capability and reduce pavement damage when constructing roads over weak soils. It serves as a separator to prevent the pavement material from punching into the subgrade under traffic. Woven and non-woven geotextiles are used for subgrade stabilization and the geosynthetic serves filtration and drainage functions in addition to separation functions.

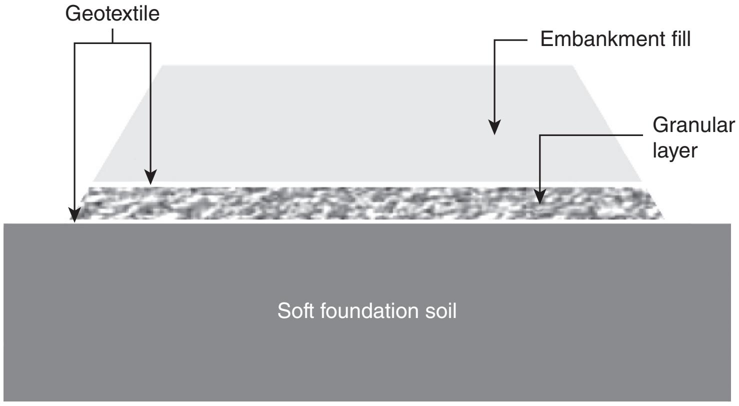

Figure 23.48 Geotextile-reinforced embankment over soft soil. (*Courtesy*: Terram Geotextiles)

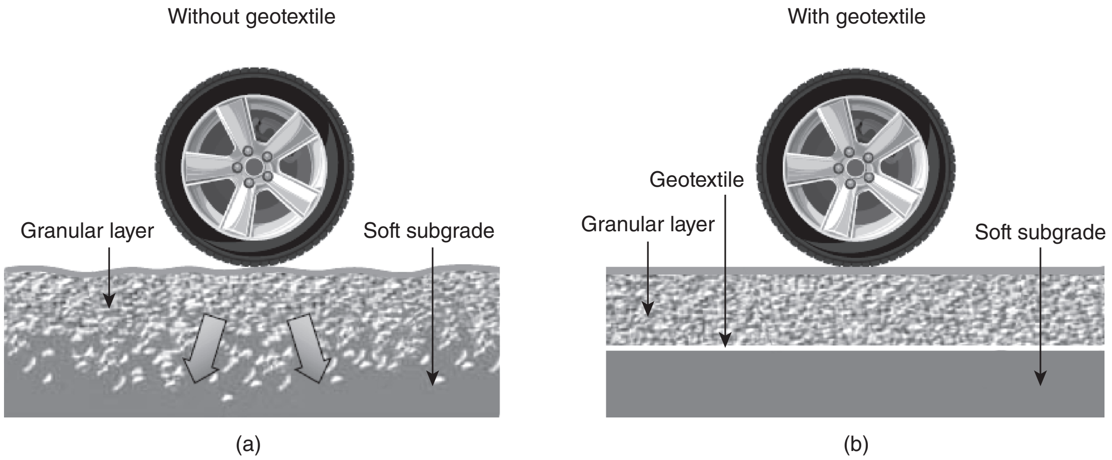

Figure 23.49 Role of geotextile in subgrade stabilization: (a) Intermixing and loss of aggregate into soft subgrade and (b) protected granular layer and pavement. (*Courtesy*: Terram Geotextiles).

Soft subgrade materials may mix with the granular base or sub-base material, as shown in Fig. 23.49(a), as a result of loads applied to the base course during construction and/or loads applied to the pavement surface that force the granular material downward into the soft subgrade or as a result of water moving upward into the granular material and carrying the subgrade material with it. The placement of a permeable geotextile between the soft subgrade and the granular material, as shown in Fig. 23.49(b), results in the following:

1. The geotextile acts as a separator to prevent the mixing of the soft soil and the granular material.
2. The geotextile acts as a reinforcement layer to resist the development of rutting.
3. The geotextile acts as a filter to allow water but not soil to pass through it.

The reinforcement application is primarily for gravel-surfaced pavements. The required thicknesses of gravel surfaced roads and airfields can been reduced because of the presence of geotextiles. Geotextiles have been used in construction of gravel roads and airfields over soft soils to solve these problems, leading to an increase in the life of the pavement and a decrease in the initial cost.

When serving as a separator, geotextiles prevent fines from migrating into the base course and/or prevent base course aggregate from penetrating into the subgrade. The soil retention properties of geotextiles are basically the same as those required for drainage or filtration. Therefore, the retention and permeability criteria required for drainage should be met. In addition, geotextiles should withstand the stresses resulting from the load applied to the pavement. Since geotextiles serve to prevent the base course aggregate from penetrating the subgrade, they must meet puncture, burst, grab, and tear strengths.

The use of geotextiles for reinforcement of gravel-surfaced roads is generally limited to use over soft cohesive soils [with California bearing ratio (CBR) < 4]. The procedure for determining the thickness requirements of the aggregate above the geotextile is based on the shear strength computed indirectly from the cone test and CBR (TM 5-818-8).

23.14.6.4 Subsurface Drainage

Geotextiles are used in subsurface drainage system as a permeable separator permitting flow of water and preventing erosion of soil along with the flowing water. Subsurface drainage is needed below highways, buildings, parking lots, shorelines, and underground construction to improve the life of the overlying structure. Non-woven geofabrics because of their high flow capacity and small pore size are commonly used for subsurface drainage. Prefabricated composites are used in place of the conventional aggregate drainage system, resulting in reduced material cost, installation time, and design complexity. Prefabricated composites consist of two components: a three-dimensional inner drain core and an attached outer fabric. The core acts as a collector and transports groundwater, while the fabric acts as a filter. Water passes through the fabric into the core. Figs. 23.50 and 23.51 show the use of geotextiles for highway and drainage.

Geotextiles have been used in toe drains of embankments, where they are easily accessible if maintenance is required and where malfunction can be detected. The primary geotextile characteristics influencing filter functions are the opening size (as related to soil retention), flow capacity, and clogging potential. These properties are indirectly measured by the apparent opening size (AOS), (ASTM D 4751), permittivity (ASTM D 4491), and gradient ratio test (ASTM D 5101). Geotextiles must also have the strength and durability to survive construction and long-term conditions for the design life of the drain.

The construction of a trench drain using a geotextile is illustrated in Fig. 23.52 and consists of the following steps:

1. The trench is excavated and the geotextile is placed ensuring intimate contact with soil surface and ensuring proper overlap after backfilling.
2. A granular bed of minimum 15-cm thickness is placed along with a collector drain.
3. The trench is backfilled, with density and strength compatible with the surrounding soil and structure.
4. Geotextile is securely overlapped with a minimum 30-cm backfill cover above the pipe. The trench is filled and compacted.

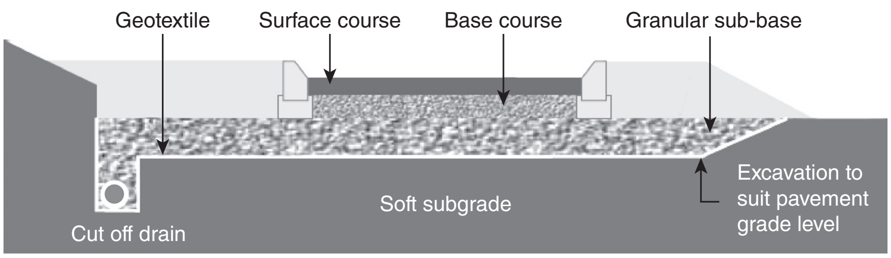

Figure 23.50 Complete layout of geotextile for pavement. (*Courtesy*: Terram Geotextiles)

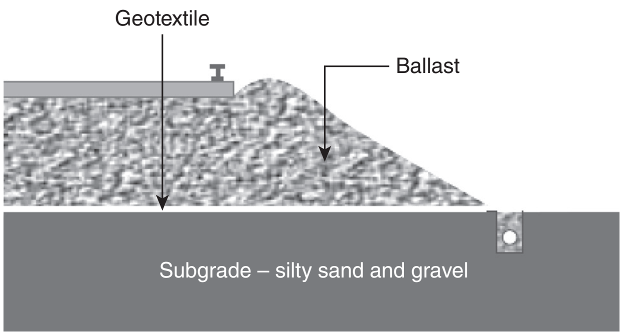

Figure 23.51 Geotextile between subgrade and ballast in railways. (*Courtesy*: Terram Geotextiles)

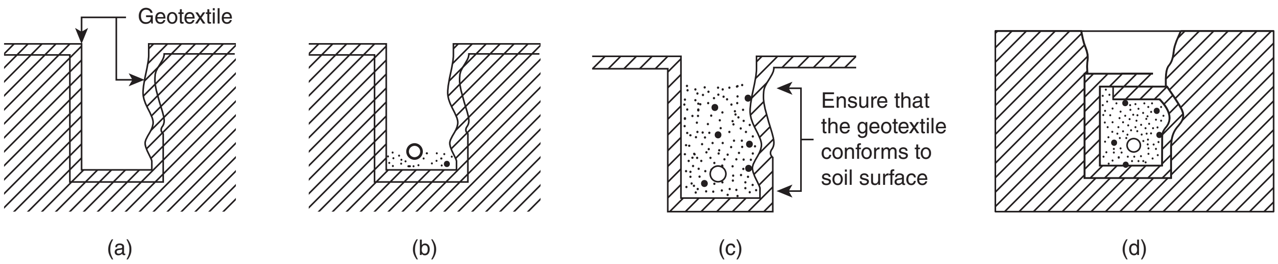

Figure 23.52 Construction of trench drain with geotextile.

23.14.6.5 Asphaltic Overlay

A geotextile helps to retard reflective cracking and improve the overlay service life when it is installed between the old and new asphaltic overlays. The geotextile absorbs the tack coat sprayed on the surface of the old pavement, forming a permanent moisture barrier. The geotextile, as a moisture barrier, protects the subgrade from water intrusion and prevents its damage. The maintenance cost of the pavement is reduced and the pavement service life is enhanced, postponing the costly resurfacing or overlay. A non-woven geotextile or geogrid is commonly used as asphaltic overlay.

23.14.6.6 Erosion Control

Geotextiles can be used for erosion control as a substitute for the conventional graded filter below the rip-rap of an earth dam. This results in substantial savings compared with using a graded filter, with far greater control during construction. Geofabrics can also be used as a substitute for rip-rap, which is also cost effective. The fabric, in the form of a mat, is positioned on the slope with nylon spacers to give the required thickness. The space between the nylon spacers and the geofabric and the earth slope is grouted with pumped structural grout. Figure 23.53 shows the erosion control function of geotextile used in embankment slope protection.

23.14.6.7 Geomembrane Protection

Geotextiles can also be used on one or both sides of a geomembrane to protect it from installation and design stresses. Geotextiles in the form of medium-to-heavy weight fabrics protect the geomembrane by acting as a cushion and minimize the chances of its puncture by sharp objects and prevent its damage from construction stresses. Geosynthetic lining systems are gaining popularity in both hazardous and non-hazardous waste landfills.

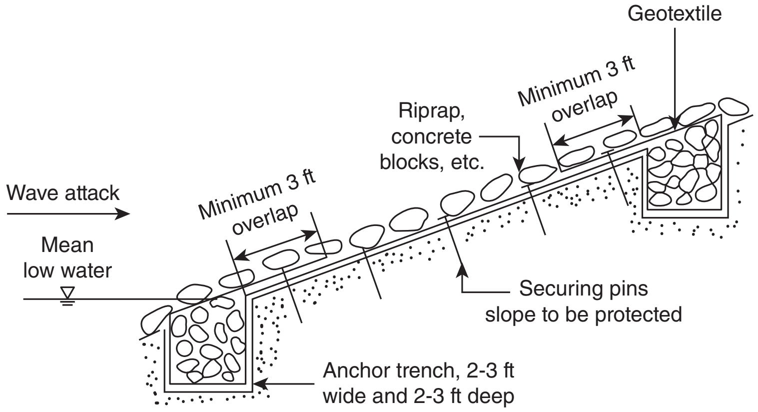

Figure 23.53 Slope protection using geotextile. (TM 5-818-8)

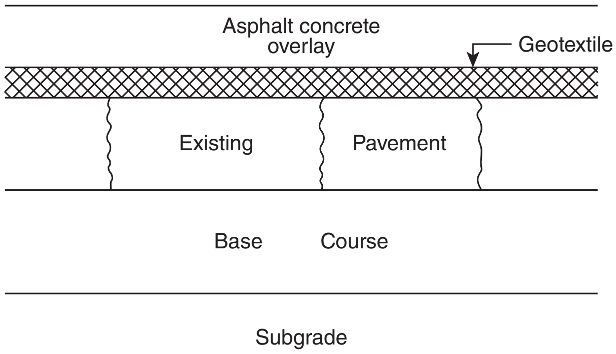

Figure 23.54 Geotextile for pavement rehabilitation. (TM 5-818-8)

23.14.6.8 Miscellaneous Applications

Paved Surface Rehabilitation

In an asphalt concrete (AC) pavement system, used in roads and airfields, the geotextile provides a stress-relieving interlayer between the existing pavement and the overlay that reduces and retards reflective cracks under certain conditions and acts as a moisture barrier to prevent surface water from entering the pavement structure. Under an AC overlay, a geotextile may provide sufficient tensile strength to relieve stresses exerted by movement of the existing pavement. The geotextile acts as a stress-relieving interlayer as the cracks move horizontally or vertically. A typical pavement structure with a geotextile interlayer is shown in Fig. 23.54. Impregnation of the geotextile with bitumen provides a degree of moisture protection for the underlying layers, whether or not reflective cracking occurs.

Reflective Crack Treatment for Pavements

Geotextiles have been successful in reducing and retarding reflective cracking in mild and dry climates when temperature and moisture changes are less likely to contribute to the movement of the underlying pavement, whereas geotextiles in cold climates have not been as successful.

23.15 Foundation Grouting

Grouting uses liquids which are injected under pressure into the pores and fissures of the ground (soil and rock). Liquid grout mixes consist of mortar, particulate suspensions, aqueous solutions, and chemical products such as polyurethane, acrylate, and epoxy. The injected grout must eventually form either a gel or a solid within the treated voids, or the grouting process must result in the deposition of suspended solids in these voids.

23.15.1 Purpose

Grouting generally is used to fill voids in the ground (fissures and porous structures) with the aim as follows:

1. To increase resistance against deformation.
2. To decrease permeability.
3. To increase cohesion, shear strength, and uniaxial compressive strength.

23.15.2 Method

Piston or screw-feed pumps deliver grout through open bore holes into fissures in rock, through lances, perforated pipes, and packered or sleeved pipes into sedimentary soils. By displacing gas or groundwater, these fluids fill the pores and fissures in the ground and thus – after setting and hardening – attribute new properties to the subsoil.

23.15.3 Grouting Equipment

The following are some of the equipment required for grouting:

1. **Drilling equipment:**
   - *Percussion drilling:* Percussion drilling produces acceptable grout holes and, generally, is the most economical method of drilling shallow holes up to a depth of 20–35 m depending on the type of rock.
   - *Rotary boring:* Rotary boring using diamond-studded coring or plug-type bits are more powerful than percussion drilling and can be used with an advantage for large depths. It is, however, costlier than percussion drilling.
2. **Grout plant:** Grout mixers with 200- to 750-L capacity have been used. High-speed colloidal-type mixers equipped with small centrifugal pumps are also in use.
3. Water meters.
4. **Agitator sumps:** After mixing, grout should be agitated to prevent settlement while it is being pumped. This can be done by pumping the grout into a sump equipped with a stirring blade.
5. **Pumps:** Hand pumps, air-driven pumps, power-driven pumps.
6. Grout lines (pipe).
7. Pressure gauges.
8. **Packers:** There are three general types of grout packers in common use: cup leather removable grout packer, mechanical packer, and pneumatic packer.

23.15.4 Applications

Grouting proves effective in the following cases:

1. When the foundation has to be constructed below the groundwater table. The deeper the foundation, the longer will be the time needed for construction. Hence, grouting will be more beneficial than dewatering.
2. When adjacent structures require that the soils of the foundation strata should not be excavated.
3. When the geometric dimensions of the foundation are complicated and involve many boundaries and contact zones.
4. When there is a need to control the groundwater flow below earth and masonry dams.

Grouting through rods, casing, or pipes may take place only at low pressures, to avoid the mix from escaping to the surface along the unsealed contact between the pipe and the ground. Fine-grained and cohesive soils are less apt to treatment with particulate grouts or chemicals. For these soils, grouting will be effective if the groutability ratio, defined by Eq. (23.25), is more than 24. As per this criterion, grouting will not be successful in fine-grained sediments with a silt content > 5% when using particulate grout with $D_{85} > 0.04$ mm.

23.15.5 Classification of Grouting

There are different types of grouting based on materials used, grouting techniques, and sequence of operations, which are as follows:

1. **Type of the ground:**
   - Grouting in soil.
   - Grouting in rock.
2. **Aim of treatment:**
   - Consolidation (strengthening).
   - Water tightening.
3. **Period of use:**
   - Temporary.
   - Permanent.
4. **Principle of the system:**
   - Permeation grouting.
   - Displacement grouting.
5. **Materials used:**
   - Portland cement grout.
   - Clay grout.

- Chemical grout.
- Asphaltic grout.

6. **Grouting technique used:**
   - Compaction grouting.
   - Soil fracture grouting.
   - Jet grouting.
   - Slurry injection.
   - Curtain grouting.
   - Blanket grouting.
   - Contact grouting.
   - Mine grouting.
7. **Sequence of operations:**
   - Stage grouting.
   - Series grouting.
   - Stop grouting.
   - Circuit grouting.
   - Soil grouting.
   - Combination methods.

23.15.6 Classification Based on Materials Used

The following are the most common types of grouts used based on materials used:

1. Cement grout.
2. Clay grout.
3. Chemical grout.
4. Asphaltic grout.
5. Precipitated grout
6. Polymerized grout

23.15.6.1 Cement Grout

Portland cement grout is a mixture of portland cement, water, and chemical and mineral additives.

1. **Types of cement for cement grout:** As per Unified Facilities Criteria, type I cement is a general-purpose cement suitable for most cement grout jobs, where the special properties of the other four types are not needed to meet job requirements. Type II cement has improved resistance to sulfate attack. Type III cement is used where early strength gains are required in grout within a period of 10 days, and it has better injectability due to its finer grind. Type IV cement develops strength at a very slow rate and is rarely used in grouting. Type V cement has a high resistance to sulfates and used occasionally in grouts if either the soil to be grouted or the groundwater at the jobsite has a high sulfate content.
2. **Fillers:** Fillers in portland cement grout are used primarily to reduce the overall cost as a replacement material where substantial quantities of grout are required to fill large cavities in rock or in soil. Sand, fly ash, diatomite, pumicite, silts, and lean clays are some of the fillers used in cement grout.

   Sand is the most widely used filler for portland cement grout. Well-graded sand passing a 1.18-mm IS sieve and minimum 15% passing a 150-μm IS sieve is used in 2:1 sand:cement proportion. Segregation can be avoided by adding more fine sand or using a mineral admixture such as fly ash, pumicite, etc.

   Fly ash is a finely divided siliceous residue from the combustion of powdered coal, and may be used both as a filler and as an admixture. The maximum amount of fly ash to be used in grout mixtures is 30% by weight of the cement to maintain strength levels comparable with those of portland cement grouts containing no fly ash.

   Diatomite is a mineral filler composed principally of silica, made up of fossils of minute aquatic plants. Small amounts of natural or processed diatomite may be used as admixtures to increase the pumpability of grout; however, large amounts as fillers will require high water/cement ratios for pumpability and justified only where low-strength grouts will fulfill the job requirements.

Pumicite, a finely pulverized volcanic ash, ash stone, pumice, or tuff, is also used as a filler in cement grout. Like fly ash and diatomite, it improves the pumpability of the mix and has a pozzolanic (hydraulic cementing) action with portland cement.

Silts and lean clays not contaminated with organic materials are sometimes used as fillers. Loess, windblown silt containing 10%–25% clay, is a suitable filler. Rock flour, a waste product from some rock-crushing operations, is also used as a filler. Grouts containing poorly graded rock flour are frequently highly susceptible to leaching.

3. **Admixtures:** Admixtures are substances that when added to portland cement grout, impart to it a desired characteristic other than bulking. Accelerators and fluidifiers are the examples.

   Accelerators cause a decrease in the setting time of grout and help to reduce the erosion of the new grout by moving groundwater, and to increase the rate of early strength gain. The most commonly used accelerator is calcium chloride. It can be added to the mixing water in amounts up to 2% of the weight of the cement. High alumina cement and plasters having a calcined gypsum base may be proportioned with portland cement to make a grout having various setting times. Other accelerators include certain soluble carbonates, silicates, and triethanolamine. When using accelerators, preliminary tests should be conducted to determine the behavior of accelerators in the grout mix.

   Fly ash and rock flour are fluidifiers or lubricants that increase the pumpability of grout. Fluidifiers and water-reducing admixtures improve the pumpability or make possible a reduction in the water/cement ratio, while maintaining the same degree of pumpability. Most of these substances are also retarders. Laboratory or field trial mixes should be batched and all pertinent effects observed and tested before adopting an unknown admixture for any project.

23.15.6.2 Clay Grout

Soils used as the primary grout ingredient include (a) the natural soils found at or near the project (b) commercially processed clay such as bentonite. Generally, where large quantities of grout are needed, local materials will be more economical. For small quantities, it is generally more economical to bring in prepared material than to set up the required mining and processing equipment to use natural soil.

1. **Selection of materials:** The use of natural soils is opted on the existence of a suitable material within a reasonable distance of the project site. Natural soils for use as a grout ingredient may be (a) fine-grained soils with low plasticity (silt and glacial rock flour) and (b) fine-grained soils of medium-to-high plasticity. The first category of soils are generally used as fillers only. The second category of soils may be used both as fillers and admixtures. The best source of soils for grouts will be alluvial, eolian, or marine deposits. Because of its ability to adsorb large quantities of water, a high percentage of montmorillonite is desirable for clay grouts. The most commonly used commercially processed clay is bentonite, a predominantly montmorillonitic clay formed from the alteration of volcanic ash. Normally, a clay with liquid limit less than 60 is not suitable for grout where a high clay mineral content and/or a high ion exchange capacity is required.
2. **Admixtures:** For the purpose of modifying the basic properties of a clay grout to achieve a required result, certain additives can be used. Portland cement can be used in clay grouts to produce a set or to increase the strength. The amount of cement required must be determined in the laboratory so that the required strength can be obtained and the grout will be stable. The presence of cement may affect the groutability of clay grouts, a point which must be considered. For large amounts of cement, portland cement grout with soil additives should be considered. Sands can be used as fillers in clay–cement grouts where voids to be filled are sufficiently large to permit intrusion of these particle sizes.
3. **Proportioning clay grout:** Once a soil has been determined suitable as a grout material for a given job, it is necessary to determine the water and admixture requirements to achieve desired properties in the grout. The grout must have sufficient flowability without excess shrinkage, and after a specified time, it should develop a gel of sufficient strength. The flowability will depend upon the water/clay ratio, which should be kept to a minimum. Trial batches with varying proportions of soil, water, and admixtures should be tested for stability, viscosity, gel time, shrinkage, and strength. From the results, the most suitable mixtures can be selected and criteria for changes in the mixture proportions to meet field conditions can be determined.

23.15.6.3 Asphalt Grout

Large subsurface flows of water are, at times, difficult to stop by grouting with cement, soil, or chemicals. For these conditions, asphalt grouting has sometimes been used successfully, particularly in sealing water courses in underground rock channels. Asphalt grout has also been used to plug leaks in cofferdams and natural rock foundations.

Asphalt is generally heated to 200°C or 230°C before injection to use for grouting. Asphalt emulsions are applied cold. In the emulsion, the asphalt is dispersed in colloidal form in water. After injection, the emulsion must be broken so that the asphalt can coagulate to form an effective grout. Special chemicals are injected with the emulsion for this purpose. Coal-tar pitch is not a desirable material for grouting, since it melts more slowly and chills more quickly than asphalt grout.

23.15.6.4 Chemical Grout

The primary advantages of chemical grouts are their low viscosity and good control of setting time. Disadvantages are the possible toxic nature of some chemicals and the relatively high cost. Because of the variety of the chemicals that can be used and the critical nature of proportioning, chemical grouts should be designed only by personnel competent in this field. Commercially available chemical grouts should be used under close consultation with the producers.

For maximum efficiency of chemical grouting, the fines in the soils must be less than 20%. The resulting product is often sandstone-like material with an unconfined compressive strength over 4100 kN/m^2.

The following are the commonly used materials for chemical grouting:

1. Sodium silicate solutions and inorganic reagents, for example, sodium aluminate, sodium bicarbonate.
2. Acrylates (Ac-400) and sodium silicate (GEOLOC-4).
3. Polyurethanes and MS silicates.
4. Activated silica liquor and an inorganic calcium-based reagent called Silascol.

The new product Silascol forms stable calcium hydrosilicates, with a crystal structure similar to cement. Medium and fine sands have been treated effectively in Italy using this product. The product also provides higher safety against pollution, as might be the case for other chemical grouts.

Chemical grouts are often used for water control purposes, owing to their low viscosity and better control of setting time. Chemical grouts fill the voids in sandy soils, making them waterproof.

23.15.6.5 Polymerized Grout

In this type of grouting, soluble monomers are mixed with suitable catalysts to produce and control polymerization and are injected into the voids to be filled. The mixture generally has a viscosity near that of water and retains it for a fixed period of time, after which polymerization occurs rapidly. Because of the low viscosity, polymer grouts can be used in soils having a permeability as low as 10^{-5} cm/s, which would include sandy silt and silty sand. The resulting product is very stable with time. The monomers may be toxic until polymerization occurs, after which there is no danger.

Some of the more common polymer-type grouts utilize the following chemicals as the basic material:

1. **Acrylamide:** Acrylamide is used in combination with methylene-bisacrylamide, which produces a polymerization that traps the added water in the gel. These grouts are expensive, but because of the low viscosity, ease of handling with recommended equipment, and excellent setting time control, they are suitable for certain applications. The ingredients are toxic and must be handled with care, but the final product is non-toxic and insoluble in water.
2. **Resorcinol-formaldehyde:** This resin-type grout is formed by condensation polymerization of dihydroxybenzene (resorcinol) with formaldehyde when the pH of the solution is changed. The final product is a non-toxic gel possessing elastic–plastic properties and high strengths when tested in a mortar form. The grout has excellent setting-time control, instantaneous polymerization, and a low viscosity prior to polymerization.
3. **Calcium acrylate:** Calcium acrylate is a water-soluble monomer that polymerizes in an aqueous solution. The polymerization reaction utilizes ammonium persulfate as the catalyst and sodium thiosulfate as the activator. The rate of polymerization is controlled by the concentration of the catalyst and the activator. The solution has a low viscosity immediately after mixing that increases with time.
4. **Epoxy resin:** Many different compounding of epoxy resins are available commercially. The epoxy is found to develop very good bond with the moist granite, was not too brittle, and the effective volume shrinkage during curing was very low.

23.15.7 Classification Based on Grouting Technique

Information on the size and continuity of groutable natural openings in soil/rock below the surface will be relatively meagre at the start of grouting operations and only slightly better after the grouting is completed. The presence of groutable voids can be ascertained before grouting and verified by grouting, but their sizes, shapes, and ramifications will be largely conjectural. The art of grouting consists of being able to satisfactorily treat these relatively unknown subsurface conditions without direct observation. Currently, there are following four types of grouting methods used:

1. Compaction grouting.
2. Fracture grouting.
3. Jet grouting.
4. Precipitation grouting.
5. Slurry injection grouting.
6. Curtain grouting.
7. Blanket grouting.
8. Contact grouting.
9. Mine grouting.

These grouting methods will be discussed in the following subsections.

23.15.7.1 Compaction Grouting

Compaction grouting is a unique grouting process that injects mortar-type grout of low slump under high pressure to densify the loose soil formations beneath distressed structures. The grout generally does not enter soil pores but remains as a homogenous mass that gives controlled displacement to compact loose soils, or controlled displacement for lifting of structures, or both. Compaction grouting is discussed as follows:

1. **Composition:** The major component of the compaction grout mix is sand or a sandy soil with 10%–30% fines. Typically, three parts of sand are used with one part cement and/or fly ash. Bentonite, of maximum 3%–5% of combined weight of cement and fly ash, is usually added to increase pumpability. The quantity of cement used varies from 0 to 300 kg/m^3 of grout. In most cases, fly ash is a desirable substitute for cement in an effort to get an appropriate fines content in the mix. Fly ash allows much more working time before setting and costs less than cement. When it is available, it is frequently used to replace 50% or more of the cement content.
2. **Grout spacing and sequence:** The majority of compaction grouting is done with pipes at spacings of 1.5–2.1 m. Area treatments with greater overburden pressure, that is, densification of loose soils created by soft ground tunnelling operation, are usually at 2.4–3 m spacing. The usual injection pressure ranges from 0.5 to 3 N/mm^2. The ground will usually show refusal when the pressure is greater than about 3 N/mm^2. Surface heave usually precedes refusal and may be taken as an indication of refusal.

   Slow pumping rates of the order of 0.01–0.02 m^3/min are used in poorly draining soils and close to the surface. Medium rates of the order of 0.02–0.1 m^3/min are used in free-draining or dry soils with reasonable cover and fast pumping rates of 0.1–0.3 m^3 or more are used in loose soils or under safe conditions involving significant cover.
3. **Construction techniques:** A primary and secondary sequencing of pipes will allow for initial densification followed by secondary densification, which is confined by the previous work. Compaction grouting can also be sequenced upward or downward within a single grout pipe. Upstage grouting or injection from the bottom up as the pipe is withdrawn is the most common system. It is less expensive and usually as effective as its counterpart, downstage grouting. A combination of the two systems utilizes “down-stage grouting” at the top of the treatment zone followed by “up-stage grouting” for all other work.

   The compaction grout mass is generally spherical (Fig. 23.55), but its shape is ultimately governed by numerous other factors including grout mix design, injection rate, the strength and texture of soil zones, the overburden or applied structural loads, etc. As the compaction grout bulb grows during injection, the soil nearest to the bulb undergoes severe deformation and stressing, resulting in some local zones of distress at the interface of the soil and grout mass. Areas further away from the interface are more uniformly compacted. In the United States, compaction grouting has clearly replaced slurry injection or “pressure grouting” as the preferred method of densification grouting (Rubright and Bandimere, 2004).

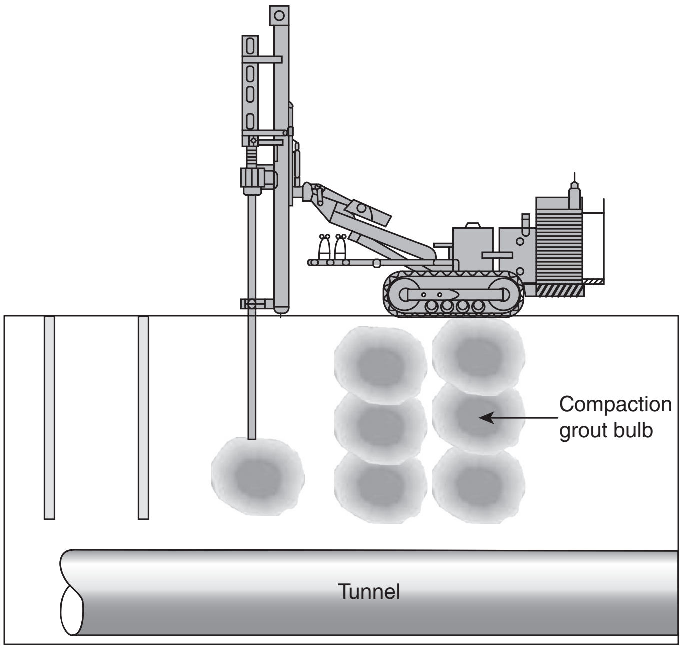

Figure 23.55 Compaction grouting to prevent settlements during tunneling.

Surface and/or structural heave is the most common limiting factor in compaction grouting. Heave indicates that compaction stresses have exceeded the confining stresses and that the soil mass is fracturing rather than compacting. Injection should proceed to the next stage once heave is observed.

4. **Applications:** In general, compaction grouting is used as a remedial measure beneath or adjacent to an existing structure:
   - To increase a bearing capacity of a soil due to load changes (modification) of an existing structure.
   - To densify loose fills.
   - To prevent settlement in loose soils caused by adjacent excavation activity, sinkhole activity, improper dewatering, broken utility lines, or change of moisture content in a collapsible soil, that is, loess.

Compaction grouting is mostly used in loose, granular soils above or below the groundwater table with SPT N-values in the range of 0–15. N-values can usually be improved by 10 or more points depending on the efficiency and effectiveness of the grouting. Compaction grouting is found to aggravate the settlements in thick saturated silt or clay strata and hence should not be used.

Compaction grouting was developed in the United States and was used in projects ranging from remedial type to densification of foundation soils prior to construction and to prevention of settlement in tunneling through soft ground. A typical example of compaction grouting was in a bottom hill subway of Baltimore, in which the settlement was prevented during excavation. Another example is the densification of a liquefiable stratum under West Pinpolis Dam in South Carolina.

23.15.7.2 Fracture Grouting

Fracture grouting or Soil fracture grouting or *soilfrac* is defined as the locally confined and controlled fracturing of a soil unit without significant effect on the soil structure using a fluid grout. The method consists of the introduction of fluid or semi-fluid material into the ground, increasing the local volume at the point of injection. Soil fracture grouting injection relies on the propagation of a neat fluid grout at pressures in excess of the hydrofracture pressure, as shown in Fig. 23.56.

Fracture grouting causes hydro-fractures in the *in-situ* soil using neat fluid grout. A sleeve port pipe is introduced into a predrilled hole beneath a foundation. The grout is injected under pressure at strategic locations

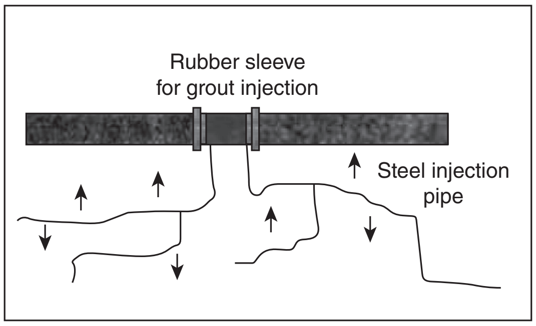

Figure 23.56 Soil fracture grouting technique. (*Courtesy*: Essler et al., 2000. With permission from ASCE.)

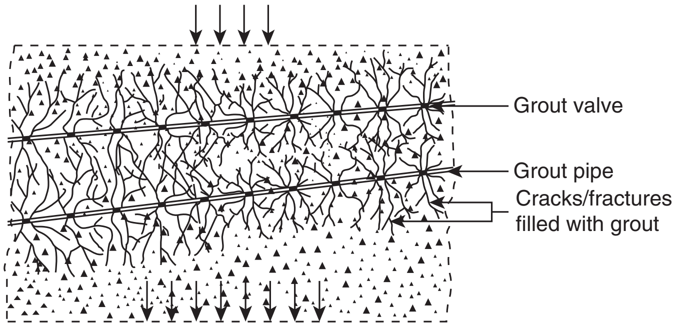

Figure 23.57 Framework for soil fracture grouting.

through the ports in the pipe in such a way that the sum of the reachable cavities in the surrounding soil accommodates only a small percentage of the amount of liquid introduced. After the fracturing pressure in the soil has been exceeded, cracks open up in the soil, which are widened immediately by the subsequent influx of grout. By injecting small amounts of solid substance in each grouting operation and by repeatedly pressurizing individual grouting valves, it is possible to achieve a grout framework of hardened solid matter in the soil mass, as shown in Fig. 23.57.

The technique has been mainly used to reduce the settlements of existing structures due to the construction of tunnels below the structure and to actually cause controlled ground heave to compensate these settlements. Soil fracture grouting is also known as compensation grouting. The technique has been used to re-level structures or to protect structures from settlement while a tunnel machine passes below.

Ground improvement by soil fracture grouting is based on the following three mechanisms:

1. The soil unit or skeleton is reinforced by a series of hard grout lenses, which propagate out from the injection point to form a matrix of hard grout and soil.
2. The fluid grout finds and fills voids and causes some compaction in the coarse-grained soil along the grout lenses created.
3. The PI of saturated clays decreases through the exchange of calcium ions originating from cement or other fillers.

The following variables control the grout composition and its design:

1. Available materials.
2. Grouting geometry.
3. Size of grout area.
4. Soil type.
5. Phase of work.

Soil fracture grouting uses some sensitive equipment that is usually not designed to operate in a dusty and dirty environment. Cased bore holes of 100-mm diameter are used for installation of grout pipe, as shown in Fig. 23.58. A wide range of grout mix composition with water-solid ratio of 0.4:1, can be used depending on the soil conditions. Large-scale trials should be carried out before actual grouting to validate the effectiveness of soil fracture grouting as well as to ascertain the influence of tunnel lining, as shown in Fig. 23.59. Continuous observation of movements of different parts of the existing structure should be done through the use of instrumentation consisting of electronic water-level systems, hydrostatic pressure leveling, automated geodetic survey, permanent vertical or horizontal inclinometers, tilt meters, etc. The instrumentation data should be represented in a three-dimensional graph using appropriate software.

The following is the typical sequence of operations in fracture grouting:

1. Site investigation.
2. Drilling.
3. Installation of soilfrac sleeve pipes.
4. Primary grouting.

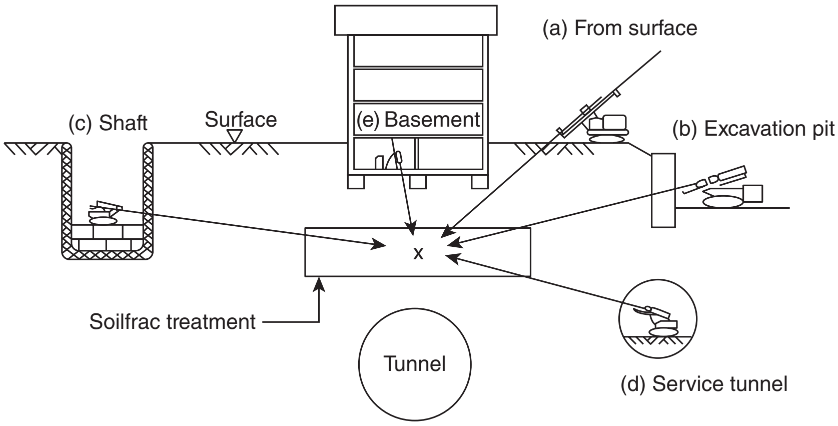

Figure 23.58 Drilling for soil fracture grouting (a) from the surface, (b) from a working trench, (c) from a shaft, (d) from existing service tunnels, and (e) from basement. (*Courtesy*: Falk and Burke, 2003. With permission from ASCE.)

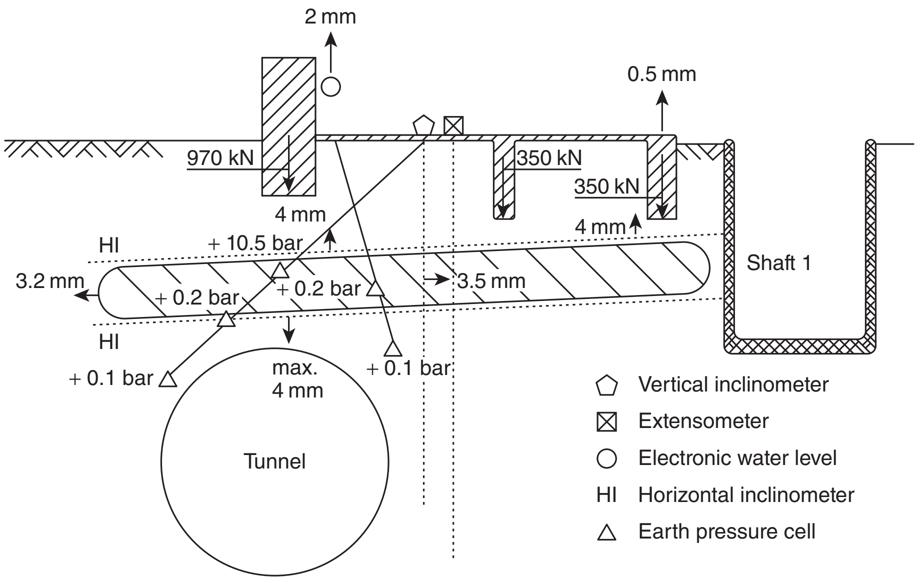

Figure 23.59 Large-scale trial for soil fracture grouting with instrumentation. (*Courtesy*: Falk and Burke, 2003. With permission from ASCE.)

5. Repeated fracture grouting.
6. Parameter acquisition.
7. Monitoring.
8. Documentation of the works and elaboration of grouting programs.

23.15.7.3 Jet Grouting

Jet grouting was introduced in Japan in the early 1970s. In this method, a grout pipe is rotated at a controlled rate from which a horizontal air or water jet at high pressure excavates a cavity. The cavity is then filled with appropriate grout. Materials used in this type of grouting are cement slurry, cement/sand mortar, chemical grouts, and cement/bentonite slurry.

The advantage of jet grouting is that it can be applied to most types of soils. The primary use is for underpinning and for cutoff walls. The disadvantage of the method is that many variables need to be controlled, namely grout mix, jet nozzle diameter, jet nozzle pressure, grout flow rate, pipe withdrawal rate, etc.

Water jets with a pressure of 30–60 MPa for an overburden soil such as silt, sand, etc., and > 200 MPa for rock formation are employed. Jet grouting requires compressed air for obtaining maximum eroding energy and for conveying spoil up to the ground surface. Compressed air may be generated by a low-pressure compressor rated at 0.7 MPa for work up to a 20-m depth (Essler and Yoshida, 2004). However, a high-pressure compressor is required to withstand the groundwater pressure for deeper works.

The strength of the treated ground is usually assessed on the basis of unconfined compressive strength tests on samples obtained by coring. The minimum required standard unconfined compressive strengths are found by experience as 1 MN/m^2 in cohesive ground and 2 MN/m^2 in granular ground for a grout water/cement ratio of 1.

Applications

Jet grouting is found to be suitable for a multitude of applications as follows:

1. *Groundwater control applications*:
   - Preventing flow through the sides or base of an excavation.
   - Controlling groundwater during tunnelling.
   - Preventing or reducing water seepage through a hydraulic structure.
   - Preventing or reducing contamination flow through the ground.
2. *Movement control applications*:
   - Prevention of ground or structure movement during excavation or tunnelling.
   - Supporting the face or sides of a tunnel during construction or in the long term.
   - Increasing the factor of safety of embankments or cuttings.
   - Providing support to piles or walls to prevent or reduce lateral movement.
3. *Support applications*:
   - Underpinning buildings during excavation or tunnelling.
   - Improving the ground to prevent failure through inadequate bearing.
   - Transferring foundation load through weak material to a competent strata.
4. *Environmental applications*:
   - Encapsulating contaminants in the ground to reduce or prevent pollution of groundwater resources.
   - Providing lateral or vertical barriers to contaminant flow.
   - Introducing reactive materials into the ground to treat specific contaminants by creating permeable reactive barriers (PRBs).

23.15.7.4 Precipitation or Permeation Grout

In this process, chemicals are mixed in liquid form for injection into a soil and a reaction between the chemicals results in precipitation of an insoluble material. Filling of the soil voids with this insoluble material results in a decrease in the permeability of the soil mass and may, for some processes, bind the particles together, with resulting strength increase.

The most common chemical used for precipitated grouting is sodium silicate, which is a combination of silica dioxide (SiO_2), sodium oxide (Na_2O), and water. The viscosity of the fluid can be varied by controlling the ratio of SiO_2 to Na_2O and by varying the water content. Silicate can be precipitated in the form of a firm gel by neutralizing the sodium silicate with a weak acid. The addition of bivalent or trivalent cations will also produce gelation.

The technique of injecting a low–viscosity, soluble grout using sodium silicate as the primary component under pressure is also called permeation grouting. Permeation grouting was principally used in sands and silty sands for soil stabilization and water control. The grout infiltrates the pore spaces between soil particles, effectively gluing the soil mass together to provide increased strength and stiffness and to decrease the permeability. Permeation grouting is most widely used in providing pre-support during tunnelling and supporting excavation, and for underpinning structures. Typically, sodium silicate (water glass) is the primary component, with either organic or inorganic reactants used to provide a predictable, controlled, and permanent gel.

Instantaneous gelling prior to injection in the soil mass is overcome by either diluting the silicate and producing a soft gel or injecting the silicate and the reactive compound separately in the ground. Alternately, the addition of an organic ester to a chemical grout results in sufficient setting time to permit adequate grout injection and a high-strength grout.

Another form of precipitation utilizes a combination of lignosulfite and bichromate (chrome lignin) which when mixed will form a firm gelatinous mass. By varying the concentration of bichromate, the setting time may be controlled to vary through a range from 10 min to 10 h. The resulting gel strength will vary depending upon the nature and concentration of lignosulfite and of chrome, and the pH of the mixture. The hexavalent chromium is toxic and requires special precaution when mixing. Also water may leach highly toxic hexavalent chromium from the gel leading to contamination of water supplies.

23.15.7.5 Slurry Injection Grouting

Slurry grouting is the intrusion of the grout material into voids and cracks underground. It is frequently used primarily to reduce the permeability of rocks beneath new dams. The grout materials used are cement, clay (bentonite), sand, additives, micro-fine cement, fly ash, lime, and water. These grouts cannot be injected into soils finer than medium-to-coarse sands. Micro-fine cement has proven to be beneficial in achieving greater permeation of the grout into shear zones than portland cement.

The suitability of slurry grouting for a given soil can be assessed by the groutability ratio, N, defined for the soil as

$$N_{85} = \frac{D_{15} \text{ of soil}}{D_{85} \text{ of grout}} \tag{23.25}$$

where D_{15} is the particle size by which 15% of the soil is finer and D_{85} the particle size by which 85% of the grout material is finer.

If $N_{85} > 24$, grouting is consistently possible. If $N_{85} < 11$, grouting is not possible. The groutability ratio may also be defined by

$$N_{95} = \frac{D_{10} \text{ of soil}}{D_{95} \text{ of grout}} \tag{23.26}$$

where D_{10} is the particle size by which 10% of the soil is finer and D_{95} the particle size by which 95% of the grout material is finer.

If $N_{95} > 5$, grouting is consistently possible. If $N_{95} < 2$, grouting is not possible.

23.15.7.6 Curtain Grouting

Curtain grouting is the construction of a curtain or barrier of grout by drilling and grouting a linear sequence of holes. Its purpose is to reduce permeability. The curtain may have any shape or attitude. It may cross a valley as a vertical or an inclined seepage cutoff under a dam; it may be circular around a shaft or other deep excavation, or it maybe nearly horizontal to form an umbrella of grout over an underground installation. A grout curtain may be made up of a single row of holes, or it may be composed of two or more parallel rows.

For grout curtains, holes are initially drilled at a large spacing of 6 to 12 m. These holes are referred to as primary holes and are grouted before any intermediate holes are drilled. Intermediate holes are located by splitting the intervals between adjoining holes; the first intermediates are midway between primary holes, and the second intermediates are halfway between primary and first intermediate holes. Spacings between holes are split in this fashion until the grout consumption indicates the rock to be satisfactorily tight. All holes of an intermediate set in any section of the grout curtain are grouted before the next set of intermediates is drilled. Although primary holes are most often drilled at 6-m spacing, other spacings are equally acceptable. If grout frequently breaks from one primary hole to another, an increase in the primary spacing is indicated. If experience in apparently similar conditions suggests that a final spacing of 1.5 to 3 m will be satisfactory, a primary spacing of 9 m may be in order since it will break down to 2.25 m with the second set of intermediates.

23.15.7.7 Blanket Grouting

In blanket grouting, also known as area grouting, the grout is injected into shallow holes drilled on a grid pattern to improve the bearing capacity and/or to reduce the permeability of broken or leached rock. Such grouting is sometimes called consolidation grouting. Blanket grouting may be used to form a grout cap prior to curtain grouting lower zones at higher pressures, or it may be used to consolidate broken or fractured rock around a tunnel or other structure underground.

23.15.7.8 Contact Grouting

Contact grouting is the grouting of voids between the walls of an underground excavation. These voids may result from excavation over break, concrete shrinkage, or a misfit of lining to the wall of the excavation. The crown of a tunnel is a common locale for contact grouting.

23.15.7.9 Mine Grouting

Grout may be used to fill abandoned mines or large natural cavities underlying engineering structures to prevent or stop roof collapse and subsidence, which is known as mine grouting. The size of these openings permits the use of a grout containing sand or sand and fine gravel. If seepage control is involved, a second or a third phase of grouting may be required with the coarser ingredients omitted from the grout to properly seal the smaller voids. Mine maps should be used, if available, to reduce the number of holes needed to inject the grout. Observation holes should be used to check the distribution of grout from various injection points. Large solution cavities, like mines, can be grouted with a coarse grout if sufficiently free from debris and muck. Since grout is unlikely to displace an appreciable amount of solution-channel filling, it may be necessary to provide access to the cavities and manually clean them prior to backfilling with concrete or grout. Cleaning is particularly important if seepage control is the purpose of the treatment.

23.16 *In-Situ* Soil Mixing

Soil mixing is a ground modification system that blends a cement grout with an *in-situ* soil to form soil–cement elements, panels, or columns. Soil mixing has the advantage of being virtually vibrationless, applicable to a wide variety of soil conditions, highly sustainable due to its use of *in-situ* materials, and fully instrumented for process control and quality assurance.

23.16.1 Types of In-Situ Soil Mixing

There are three types of soil mixing techniques to provide ground improvement solution:

1. Cutter soil mixing (CSM).
2. Jet mixing (JM).
3. Deep soil mixing (DSM).

Selection of the appropriate technique depends on several factors, including the object of ground treatment, the characteristics of the soil, and the desired end result. CSM is suitable for all types of soils. JM is suitable for sands, silts, and clays but less suitable for gravels than CSM. DSM is suitable for sands and soft silts and clays. It is less suitable than CSM for gravels and may not be suitable for stiff silts and clays.

23.16.1.1 Cutter Soil Mixing

CSM was developed from the diaphragm wall technology and utilizes two sets of counter-rotating, vertically mounted cutter wheels. The wheels cut the surrounding soil while blending the injected slurry with the *in-situ* soil to form soil–cement panels, 0.5–1.2 m in width.

The CSM system allows for control of the speed and rotational direction of the cutter wheels; each wheel is controlled independently. Instrumentation housed directly inside the cutter head relays the x, y, and z coordinates of the mixing tool in real time to a computer display inside the operator's cab. This provides assurance of complete overlap between panels to depths of up to about 40 m. Figure 23.60 shows the sequence of operations in a typical ground improvement project using CSM.

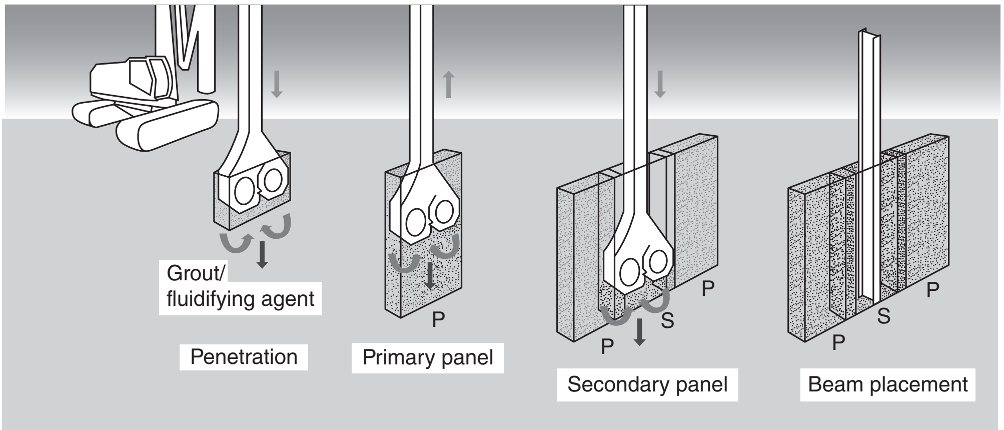

Figure 23.60 Cutter soil mixing. (*Courtesy*: Malcolm Drilling Company, USA)

As a result of the hydraulic motor being housed inside the cutter wheels themselves, energy is applied directly at the point of attack and not at the top of the Kelly bar. The wheels can be equipped with rock teeth to allow cutting through difficult soils, including cobbles up to 20 cm in diameter or bedrock with an unconfined compressive strength of up to 35 MPa.

Following are the benefits of CSM:

1. CSM can be used for difficult soils, including stiff plastic clays, gravels, and cobbles.
2. Building of rectangular panels, not columns, make it ideally suited for constructing walls.
3. Instrumentation inside the cutting head monitors x, y, and z coordinates of the tool in real time.
4. The cutting and mixing energy is generated at the point of attack.
5. It has the ability to key into the underlying bedrock.

23.16.1.2 Jet Mixing

JM is a unique soil mixing system that combines mechanical paddle mixing with high-pressure hydraulic energy to shear and blend the soil *in-situ*, creating high strength soil–cement mix. The high-energy JM system allows to extend soil mixing to stiff, highly plastic clays and weathered rock.

JM is fast and it is possible to construct cylindrical soil–cement elements, 0.75–1.0 m in diameter, to depths of up to 20 m. It is ideally suited to construct soil–cement piers for settlement control and improve bearing capacity. JM can provide permeability of 1×10^{-6} cm/s or less for cutoff walls and groundwater barriers.

1. **Benefits of JM:**

   Following are the benefits of JM:

   - Combination of mechanical and hydraulic energy enables efficient installation, even in stiff, highly plastic clays.
   - High-energy system results in uniform, high-strength soil–cement blend.
   - It is an economical system to construct walls, permeability barriers, and soil–cement columns.
   - It has the ability to key into the underlying bedrock.

23.16.1.3 Deep Soil Mixing

DSM system utilizes mechanical mixing tools to shear the soil *in-situ* and mix it with a cement slurry pumped at low pressure. This method has the ability to create large soil mix columns, typically 1.5–2.4 m in diameter to depths of up to 24 m.

DSM is an economical system for mass ground improvement projects. It is ideally suited to provide settlement control and improve bearing capacity in soft soils. Its ability to create large-diameter soil mix columns also make it well suited to mitigate the effects of liquefaction-induced settlements and lateral spreading. DSM is also the soil mixing method of choice for *in-situ* remediation and encapsulation of contaminated soils and for the construction of PRB walls.

Following are the benefits of DSM:

1. It has the ability to construct large-diameter columns.
2. It is a very efficient and cost-effective method to mix large volumes of soil.
3. It is an effective method for settlement control, liquefaction mitigation, and remediation of contaminated soils.

23.16.2 Benefits of Soil Mixing

Following are the benefits of *in-situ* soil mixing:

1. It uses soil–cement mixed *in-situ*, eliminating the use of concrete or aggregate.
2. It is economical system to construct deep foundations and retaining walls.
3. It is useful to improve a wide range of soils.
4. It is an excellent method for liquefaction mitigation, settlement control, excavation support, and seepage control.
5. The method can solidify contaminants in place.

23.16.3 Soil Mixing Applications

In-situ soil mixing is used in diverse marine and land applications as follows:

1. Mitigation of liquefaction potential/seismic-induced settlement/lateral spreading.
2. Massive ground improvement for settlement control or increase in bearing capacity.
3. Excavation support/construction of temporary or permanent retaining walls.
4. Construction of cutoff walls for control of groundwater and contaminants.
5. *In-situ* encapsulation of pollutants/chemical treatment of contaminants.
6. Structural support for area loads and point loads.

23.17 Seepage Control and Dewatering Systems

Construction of buildings, powerhouses, dams, locks, and many other structures requires excavation below the water table into water-bearing soils. Such excavations require lowering the water table below the slopes and bottom of the excavation to prevent raveling or sloughing of the slope and to ensure dry, firm working conditions for construction operations. Most problems encountered in deep excavations have direct or indirect relation with groundwater. The success of excavation and construction for a structure depends, therefore, on the effectiveness in dealing with groundwater.

Dewatering is frequently required and used in granular soils as submergence of excavations occurs due to their high permeability. When groundwater levels are high in such soils, quick conditions are developed, leading to the loss of shear strength and heaving of the excavation bottom. To avoid such problems, comprehensive dewatering systems are to be designed before or during excavation. Thus, the objectives of dewatering are as follows:

1. To keep the excavation bottom dry to facilitate hassle-free and timely construction of the foundation and the structure.
2. To stabilize the banks of the excavation, thus avoiding the hazards of slides and sloughing.
3. To prevent sand boil or quick conditions on the excavation bottom and its upheaval.
4. To prevent uplift pressures on the foundation and basement which will cause uneven post-construction settlement of the structure.

23.17.1 Types of Seepage Control/Dewatering Systems

Dewatering systems may be temporary or permanent. Temporary dewatering systems are more common and are used to ensure hassle-free and timely construction of foundations and structures. Permanent dewatering systems are used for structures located below the water table and are less common. There are various types of dewatering, as given in the following list:

1. Surface water control methods such as open sumps, ditches, training walls, and embankments.
2. Gravity drainage method.
3. Well point systems:
   - Conventional well point system.
   - Vacuum well point system.
   - Jet eductor well point systems.
4. Deep well point systems.
5. Vertical sand drains.
6. Electro-osmosis.
7. Cut-off walls:
   - Cement and chemical grout curtains.
   - Slurry walls.
   - Steel sheet piling.
8. Ground freezing.

The gravity drainage method involves simple pumping equipment and may be used for relatively impermeable soils, especially on sloping sites. Other methods are described in detail in the following sections.

23.17.2 Open Sumps and Ditches

An elementary dewatering procedure involves the installation of open sumps, ditches and trench drains within an excavation from which water entering the excavation can be pumped. A sump is merely a square or rectangular pit in the ground from which water is pumped for the purpose of removing water from the adjoining area (Fig. 23.61). Sumps are used with ditches leading to them in large excavations up to a maximum of 8 m depth below the pump installation level. For greater depths, a submersible pump is required.

23.17.2.1 Dewatering in Open Excavations

Unsupported excavations in silts and fine sands may require the necessary safe side slopes. The method is more suitable for gravels and coarse sands. It may not be suitable for fine-grained soils, which may be easily removed from the ground, and soils containing a large percentage of fines are also not suitable.

Disadvantages of a sump dewatering system are as follows:

1. The method is slow because excavation and dewatering have to be done sequentially in stages.
2. Since dewatering is done only after excavation in each stage, potentially wet conditions exist during excavation.

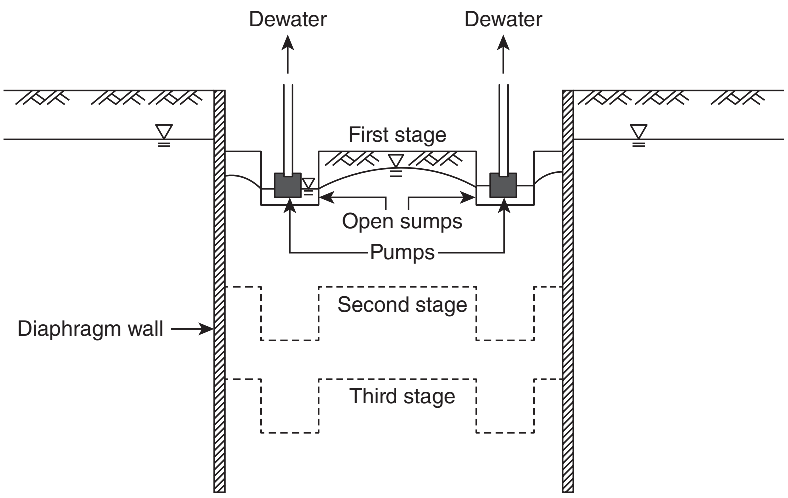

Figure 23.61 Open ditch and sump. (*Courtesy*: ebooks.narotama.ac.in)

3. Space required in the bottom of the excavation for drains, ditches, sumps, and pumps.
4. The frequent lack of workmen who are skilled in the proper construction or operation of sumps.

23.17.2.2 Dewatering in Sheeted Excavation

In this method, wooden or steel sheeting is first driven into the ground around the sump area for the full depth, the soil is excavated and simultaneously the sheeting is supported by a bracing system. The water that enters the sheeted area is pumped out, and a cage is installed inside the sump. The cage consists of wire mesh with internal strutting or a perforated pipe and the space outside and bottom of the cage is filled with filter material as shown in Fig. 23.62.

The advantage of the method is that the heaving of the bottom of excavation in sand may be reduced if sheeting is driven to the depth of an underlying impervious stratum, which reduces the seepage towards the bottom of excavation.

An inverted sand and gravel filter may be used at the bottom of excavation to facilitate construction and pumping out the seepage water. The disadvantage of the method is same as that in open excavations, i.e., existence of wet conditions during excavation.

23.17.3 Well Point System

Well point systems are the commonly used dewatering methods as they are applicable to a wide range of excavations and groundwater conditions.

23.17.3.1 Conventional Well Point System

A well point is a 5.0- to 10-cm diameter pipe 0.6- to 1.5-m long which is perforated and covered with a screen. It is made of brass or stainless steel mesh, slotted brass, or a plastic pipe. The lower end of the pipe has a driving head with water holes for jetting (Fig. 23.63). Well points are connected to 5.0- to 7.5-cm-diameter pipes known as riser pipes and are inserted into the ground by driving or jetting (Fig. 23.64). The upper ends of the riser pipes lead to a header pipe, which, in turn, is connected to a pump. The groundwater is drawn by the pump from the well points through the header pipe and discharged. Well points are usually installed with 0.75- to 3-m spacing. They may or may not be surrounded with a filter depending upon the type of soil drained.

One or more supplementary vacuum pumps may be added to the main pumps where additional air handling capacity is required or desirable. Generally, a stage of well points (well points connected to a header at a common elevation) is capable of lowering the groundwater table by about 4.5 m. Lowering the groundwater more than 4.5 m generally requires a multi-stage installation of well points, as shown in Fig. 23.65. In a multi-stage well point system, a separate pump is used for each stage of the well point system.

A well point system is usually the most practical method for dewatering, where the site is accessible and where the excavation and water-bearing strata to be drained are not too deep. For large or deep excavations, where the depth of excavation is more than about 10 m, or where the artesian pressure in a deep aquifer must be reduced, it may be more practical to use eductor-type well points or deep wells with turbines or submersible pumps, using well points as a supplementary method of dewatering if needed. Well points are more suitable than deep wells, where the submergence available for the well screens are small and close spacing is required to intercept seepage.

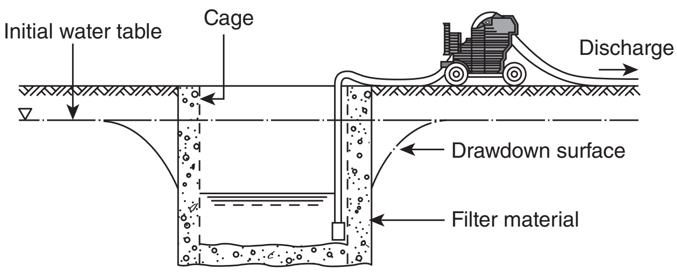

Figure 23.62 Large sump without ditch.

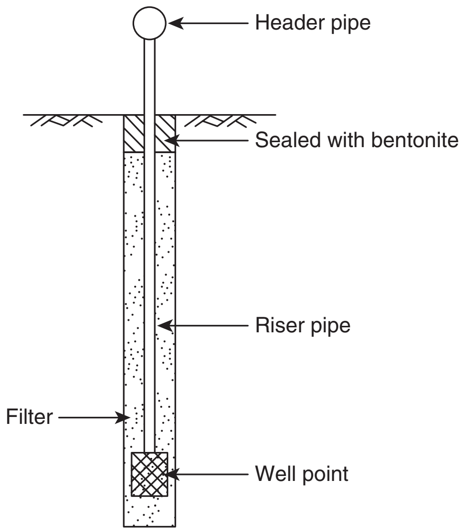

Figure 23.63 Well point.

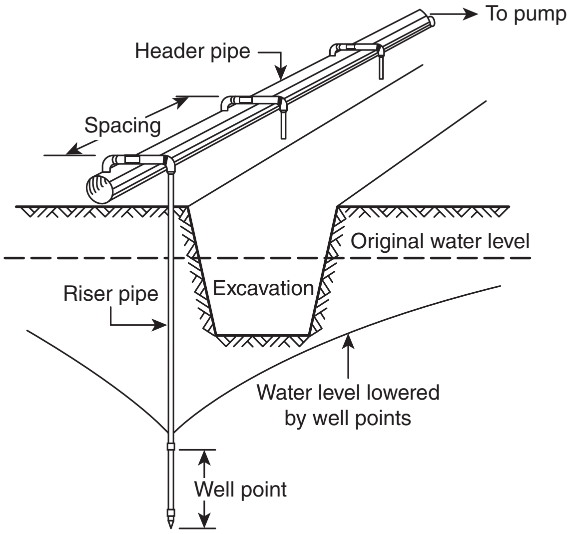

Figure 23.64 Single-stage well point system.

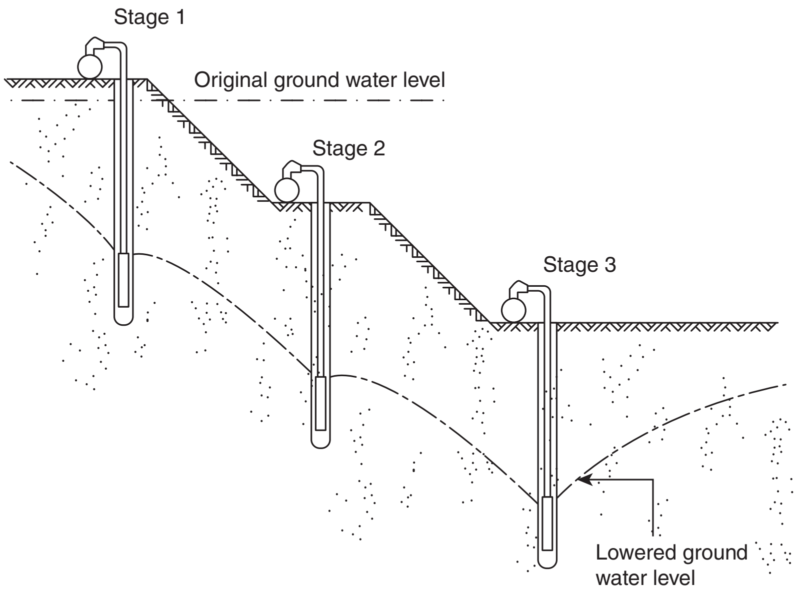

Figure 23.65 Multi-stage well point system.

23.17.3.2 Vacuum Well Point System

Silts and sandy silts ($D_{10} \leq 0.05$ mm) with a low coefficient of permeability ($k = 1 \times 10^{-5}$ to 1×10^{-3} cm/s) cannot be drained successfully by gravity methods, but such soils can often be stabilized by a vacuum well point system. A vacuum well point system is essentially a conventional well system in which a partial vacuum is maintained in the sand filter around the well point and the riser pipe. This vacuum will increase the hydraulic gradient producing flow to the well points and will improve drainage and stabilization of the surrounding soil. For a well point system, the net vacuum at the well point and in the filter is the vacuum in the header pipe minus the lift or length of the riser pipe. Therefore, relatively little vacuum effect can be obtained with a well point system if the lift is more than about 4.5 m. If there is much air loss, it may be necessary to provide additional vacuum pumps to ensure maintaining the maximum vacuum in the filter column. The required capacity of the water pump is, however, small.

23.17.3.3 Jet-Eductor Well Point System

A jet eductor pump or thermocompressor is a type of pump (Fig. 23.66) that uses the Venturi effect of a converging–diverging nozzle to convert the pressure energy of a motive fluid to velocity energy that creates a low-pressure zone that draws in and entrains a suction fluid.

This system, also known as the "jet eductor system" or "ejector system" or "eductor well point system," is similar to the well point system. Instead of employing a vacuum to draw water to the well points, the eductor system uses high-pressure water and riser units, each about 3–4 cm in diameter. A high-pressure supply main feeds water through a venturi tube immediately above the perforated well screen, creating a reduction in pressure that draws water through the large-diameter rise pipe. The high-pressure main feeds off the return water.

Jet eductor well points are installed in the same manner as conventional well points, generally with a filter, as required by the foundation soils. The two riser pipes are connected to separate headers, one to supply water under pressure to the eductors and the other for return of flow from the well points and eductors. The advantage of the eductor system is that in operating many well points from a single pump station, the water table can be lowered in one stage from depths of 10–45 m. Jet eductor well point systems are most advantageously used to dewater deep excavations, where the volume of water to be pumped is relatively small because of the low permeability of the aquifer, as in silty-to-fine sand formations.

23.17.4 Deep Well Systems

In this system, pumps are placed at the bottom of the wells and the water is discharged through a pipe connected to the pump and run up through the well hole to a suitable discharge point. A deep well system is more powerful than a well point system and requires fewer well holes at wider spacing. Deep wells can be used to dewater pervious sand or rock formations or to relieve the artesian pressure beneath an excavation. They are particularly suited for dewatering large excavations requiring high rates of pumping and for dewatering deep excavations for dams, tunnels, locks, powerhouses, and shafts.

A cased bore hole is sunk using drilling or boring rigs to a depth lower than the required dewatered level. The diameter of bore hole is 15–20 cm larger than the well inner casing, which in turn is sized to accept the submersible pump. The inner well casing has a perforated screen (Fig. 23.67) over the depth, requiring dewatering, and terminates below in 1 m of the unperforated pipe. After the slotted PVC or metal well screen (casing) has been installed, it is surrounded by backfill over the unperforated pipe length and with graded filter material over the perforated length as the outer casing is progressively withdrawn.

Excavations and shafts as deep as 90 m can be dewatered by pumping from deep wells with turbine or submersible pumps. The principal advantage of deep wells is that they can be installed around the periphery of an excavation and thus leave the construction area unencumbered by dewatering equipment and the excavation can be pre-drained for its full depth.

Deep wells may be used in conjunction with a vacuum system to dewater small, deep excavations for tunnels, shafts, or caissons sunk in relatively fine-grained or stratified pervious soils or rock below the groundwater table. The addition of a vacuum to the well screen and the filter will increase the hydraulic gradient to the well and

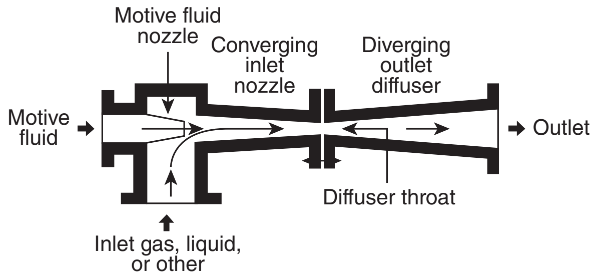

Figure 23.66 Jet-eductor pump.

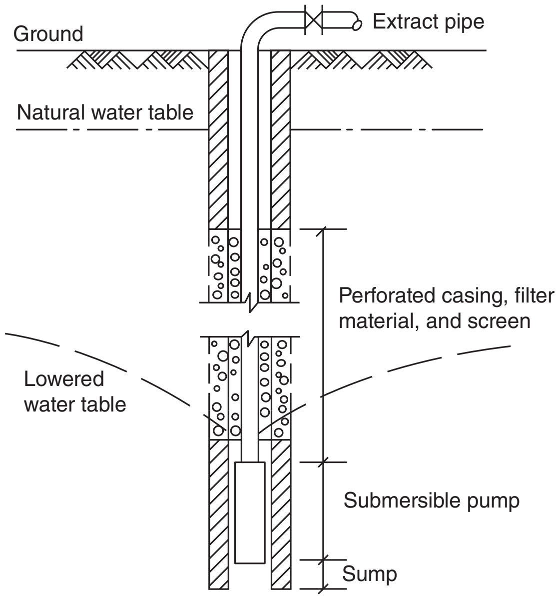

Figure 23.67 Deep-well system.

will create a vacuum within the surrounding soil that will prevent or minimize seepage from perched water into the excavation. Installations of this type require adequate vacuum capacity to ensure efficient operations of the system.

Deep well systems are of use in gravels to silty fine sands and in water-bearing rocks. They are primarily used for deep excavations and where the artesian water is present below an impermeable stratum. If this type of installation is to be designed economically, the ground permeability must be assessed from full-scale pumping tests. Because of their depth and the usually longer pumping period, these installations are more likely to cause settlement of nearby structures, and the use of recharge methods may have to be considered. Deep wells can be used for a wide range of flows by selecting pumps of appropriate size.

23.17.5 Vertical Sand Drains

Where a stratified semi-pervious stratum with a low vertical permeability overlies a pervious stratum and the groundwater table has to be lowered in both strata, the water table in the upper stratum can be lowered by means of sand drains. If properly designed and installed, sand drains will intercept seepage in the upper stratum and conduct it into the lower, more permeable stratum being dewatered with wells or well points.

Sand drains consist of a column of pervious sand placed in a cased hole, either driven or drilled through the soil, with the casing subsequently removed. The capacity of sand drains can be significantly increased by installation of a slotted 2.5- or 5-cm-diameter pipe inside the sand drain to conduct the water down to the more pervious stratum. More detailed consideration of sand drains is given in Section 23.9 of this chapter.

23.17.6 Electro-Osmosis

Soils of low permeability such as silts, clayey silts, peat etc., which cannot be dewatered by pumping from well points or wells, can be more effectively drained by well points combined with a flow of direct electric current through the soil toward the wells. In electro-osmosis, direct current is applied from anodes (steel rods) to cathodes (well points). This causes the water contained in the soil voids to migrate from the positive electrode (anode-steel rods) to the negative electrode (cathode-well point). The water that migrates to the cathode can be removed by either vacuum or eductor pumping (Fig. 23.68). Electro-osmosis is very costly and used only for difficult dewatering applications in low permeability soils, where other methods cannot be used. Electro-osmosis should never be used until a test of a conventional system of well points, wells with vacuum, or jet-eductor well points has been attempted.

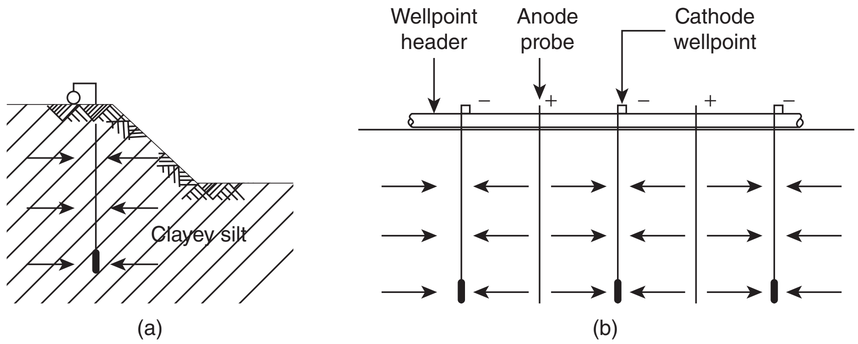

Figure 23.68 Electro-osmosis: (a) Cross section and (b) longitudinal section. (UFC-3-220-5)

23.17.7 Cutoffs

Cutoff curtains can be used to stop or minimize seepage into an excavation, where the cutoff can be installed down to an impervious formation. Such cutoffs can be constructed by driving steel sheet piling, grouting the existing soil with cement or a chemical grout, excavating by means of a slurry trench and backfilling with a plastic mix of bentonite and soil, installing a concrete wall, or freezing. However, groundwater within the area enclosed by a cutoff curtain, or leakage through or under such a curtain, will have to be pumped out with a well or well point system, as shown in Fig. 23.69.

23.17.7.1 Cement and Chemical Grout Curtains

A cutoff around an excavation in coarse sand and gravel or porous rock can be created by injecting cement or a chemical grout into the voids of the soil. For grouting to be effective, the voids in the rock or soil must be large enough to accept the grout, and the holes must be close enough together so that a continuous grout curtain is obtained. The type of grout that can be used depends upon the size of voids in the sand and gravel or rock to be grouted. Grouts commonly used for this purpose are cement and water; cement, bentonite, an admixture to reduce surface tension, and water; silica gels; or a commercial product. Generally, grouting of fine or medium sand is not very effective for blocking seepage. A single line of grout holes is also generally ineffective as a seepage cutoff; three or more lines are generally required.

23.17.7.2 Slurry Walls

A cutoff to prevent or minimize seepage into an excavation can also be formed by digging a narrow trench around the area to be excavated and backfilling it with an impervious soil. Such a trench can be constructed in almost any

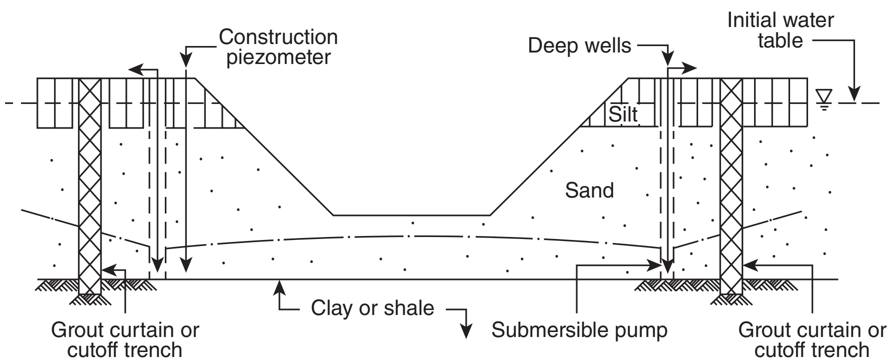

Figure 23.69 Grout curtain or cutoff trench. (UFC-3-220-5)

soil, either above or below the water table, by keeping the trench filled with a bentonite mud slurry and backfilling it with a suitable impervious soil. Generally, the trench is backfilled with a well-graded clayey sand gravel mixed with bentonite slurry.

23.17.7.3 Steel Sheet Piling

When constructing small structures in open water, it may be desirable to drive steel sheet piling around the structure, excavate the soil underwater, and then tremie in a concrete seal. The concrete tremie seal must withstand uplift pressures, or pressure relief measures must be used. In restricted areas, it may be necessary to use a combination of sheeting and bracing with wells or well points installed just inside or outside of the sheeting.

The effectiveness of sheet piling driven around an excavation to reduce seepage depends upon the perviousness of the soil, the tightness of the interlocks, and the length of the seepage path. Some seepage through the interlocks should be expected. Sheet piling is not very effective in blocking seepage where boulders or other hard obstructions may be encountered because of driving out of interlock.

23.17.8 Selection of Dewatering System

The following are the factors which influence the selection of a suitable dewatering system:

1. The location, type, size, and depth of the excavation.
2. Thickness, stratification, and permeability of the foundation soils below the water table.
3. Potential damage resulting from failure of the dewatering system.
4. The cost of installation and operation of the system.

The cost of a dewatering method or system depends upon the following:

1. Type, size, and pumping requirements of the project.
2. Type and availability of power.
3. Labor requirements.
4. Duration of required pumping.

Major factors affecting the selection of dewatering and groundwater control systems are discussed in the following list:

1. **Type of excavation:** Small open excavations, or excavations where the depth of water table lowering is small, can generally be dewatered most economically and safely by means of a conventional well point system. If the excavation requires that the water table or artesian pressure is to be lowered by more than about 6 m, a system of jet educator–type well points or deep wells may be more suitable. Well points, deep wells, or a combination thereof can be used to dewater an excavation surrounded by a cofferdam. Where groundwater lowering is expensive and where cofferdams are required, caisson construction may be more economical. Caissons are being used more frequently even for small structures.
2. **Geologic and soil conditions:** If the soil below the water table is a deep, more or less homogeneous, free-draining sand, it can be effectively dewatered with either a conventional well or a well point system. If, on the other hand, the formation is highly stratified, or the saturated soil to be dewatered is overlain by an impervious stratum of clay, shale, or rock, well points or wells on relatively close centers may be required.
   For deep aquifers, a deep-well system will generally be more applicable, or the length of the well points should be increased and the well points set deep and surrounded with a high-capacity filter.
   Some gravels and rock formations may be so permeable that a barrier to flow, such as a slurry trench, grout curtain, sheet pile cutoff, or freezing, may be necessary to reduce the quantity of flow to the dewatering system to reasonable proportions.
   Drainage of sandy silts and silts will usually require the application of additional vacuum to well or well point dewatering systems, or possibly the use of the electro-osmotic method of dewatering where soils are silty or clayey.
3. **Depth of groundwater lowering:** If the drawdown required is large, deep wells or jet eductor well points may be the best because of their ability to achieve large drawdowns from the top of an excavation, whereas many stages of well points would be required to accomplish the same drawdown.

4. **Required rate of pumping:** Turbine or submersible pumps for pumping deep wells are available in sizes from 3 to 14 in. with capacities ranging from 20 to 20000 L/min at heads up to 45 m. Well point pumps are available in sizes from 6 to 12 in. with capacities ranging from 2000 to 20000 L/min depending upon vacuum and discharge heads. Jet eductor pumps are available that will pump from 10 to 75 L/min for lifts up to 30 m.
5. **Intermittent pumping:** Pumping labor costs can occasionally be materially reduced by pumping a dewatering system only one or two shifts per day. It can be economical where the dewatered area is large; subsoils below subgrade elevation are deep, pervious, and homogeneous; and the pumping plant is oversize. This type of pumping plant operation should be permitted only where adequate piezometers have been installed and are read frequently.
6. **Effect of groundwater lowering on adjacent structures and wells:** Lowering the groundwater table increases the load on foundation soils below the original groundwater table, leading to settlement of adjacent structures within the radius of influence of a dewatering system. The possibility of such settlement should be investigated before a dewatering system is designed.

Recharge of the groundwater may be necessary to reduce or eliminate distress to adjacent structures, or it may be necessary to use positive cutoffs to avoid lowering the groundwater level outside of an excavation. Positive cutoffs include soil freezing and slurry cutoff techniques. Observations should be made of the water level in nearby wells before and during dewatering to determine any effect of dewatering.

23.18 Freezing

Ground freezing is the use of refrigeration to convert *in-situ* pore water to ice. The ice then acts as a cement or glue, bonding together adjacent particles of soil or blocks of rock to increase their combined strength and make them impervious. However, freezing is expensive and requires expert design, installation, and operation. If the soil around the excavation is not completely frozen, seepage can cause rapid enlargement of a fault (unfrozen zone) with consequent serious trouble, which is difficult to remedy.

23.18.1 Principle

The principle of ground freezing is to change the water in the soil into a solid wall of ice, which is completely impermeable. Ammonium brine or liquid nitrogen may be used as refrigerants. The principle of ground freezing is analogous to pumping groundwater from wells. To freeze the ground, a row of freeze pipes are placed vertically in the soil and heat energy is removed through these pipes. Isotherms (an isotherm is a line connecting geographical points of equal temperature) move out from the freeze pipes with time similar to groundwater contours around a well.

23.18.2 Method

The most common freezing method is by circulating brine (a strong saline solution, such as calcium chloride). Chilled brine is pumped down a drop tube to the bottom of the freeze pipe and flows up the pipe, drawing heat from the soil (Fig. 23.70). A 60-t portable refrigeration unit is typically used for ground freezing. The liquid nitrogen (LN_2) process has been applied successfully to ground freezing but its cost per unit of heat extracted is much higher than with circulated brine. Nevertheless, for small, short-term projects, particularly in emergencies, the method can occasionally be competitive.

Once the earth temperature reaches 32°F (0°C), water in the soil pores turns into ice. Then, further cooling proceeds. The groundwater in the pores readily freezes in granular soils, such as sands. Saturated sand achieves excellent strength at only a few degrees below the freezing point. If the temperature is lowered further, the strength increases marginally. In cohesive soils, such as clays, the groundwater is molecularly bonded at least in part to the soil particles. If soft clay is cooled down to freezing temperature, some portions of its pore water begin to freeze and this causes the clay to stiffen. With further reduction in temperature, more pore water freezes and consequently more strength gain is achieved. When designing for frozen earth structures in cohesive soils, it may be necessary to specify substantially lower temperatures to achieve the required strength, than in cohesionless soils. A temperature

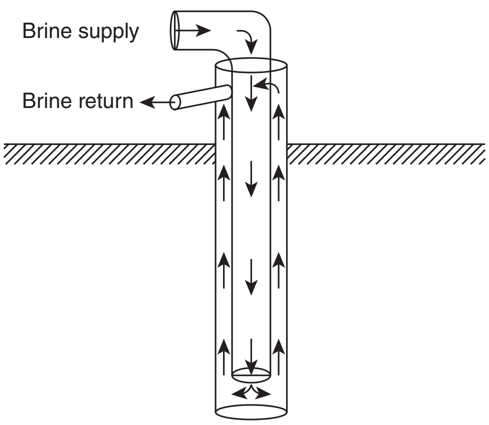

Figure 23.70 Freeze pipe detail.

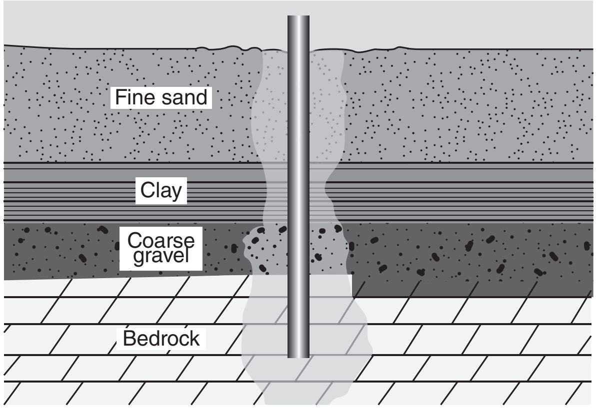

Figure 23.71 Formation of frozen earth barrier in different soils. (*Courtesy*: Nemati, 2007)

of +20°F (–7ºC) may be sufficient in sands, whereas temperatures as low as –20°F (–29ºC) may be required in soft clays.

The design of a frozen earth barrier is governed by the thermal properties of the underlying soils and the related response to the freezing system. The formation of a frozen earth barrier develops at different rates depending on the thermal and hydraulic properties of each stratum. Typically, rock and coarse-grained soils freeze faster than clays and silts (Fig. 23.71).

If heat extraction is continued at a high rate, the thickness of the frozen wall will expand with time. Once the wall has achieved its design thickness, the freeze plant is operated at a reduced rate to remove the heat flowing toward the wall, to maintain the condition.

23.18.3 Applications

The freezing method is remarkably versatile, and with ingenuity, it can be adapted to a many project conditions. The penetration of a freeze does not vary greatly with permeability, so it is much more effective as a cutoff than grout. In stratified soils, cutoffs by freezing encounter fewer problems than drainage by dewatering. Freezing can perform the dual function of a water cutoff and an earth support, eliminating sheeting and bracing. Figure 23.72 shows a circular excavation supported by a freeze wall.

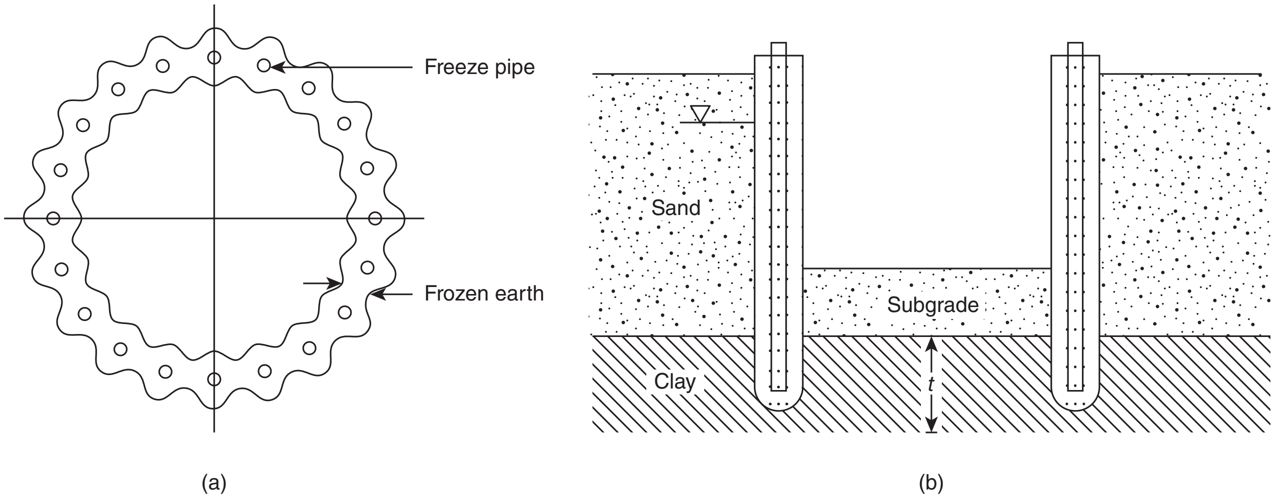

Figure 23.72 Circular excavation support by a freeze wall. (*Courtesy*: Nemati, 2007)

The following are the typical applications of ground freezing technique:

1. To provide temporary underpinning.
2. To provide temporary support for an excavation.
3. To prevent groundwater flow into the excavated area.
4. Temporary slope stabilization.
5. Temporary containment of toxic/hazardous waste contamination.
6. To stabilize earth for tunnel excavation.
7. To arrest landslides.
8. To stabilize abandoned mineshafts.

The method is obviously more relevant in relatively cold regions, where atmospheric as well as ground temperatures are low for most part of the year. Ground freezing is very costly and used only for difficult dewatering applications only where other methods cannot be used.

23.19 Heating

Heating or vitrifaction breaks the soil particle down to form a crystalline or glass product. The method uses electrical current to heat the soil and modify the physical characteristics of the soil. Heating soils permanently alters the properties of the soil. Depending on the soil, temperatures can range between 300°C and 1000°C. The impact on adjacent structures and utilities should be considered when heating is used. Applications include the following:

1. Immobilization of radioactive or contaminated soil.
2. Densification and stabilization.

Summary

Today the construction of any structure has to be taken up at the given site due to scarcity and high value of land. When the soil at the site is weak or highly compressible, one of the options is to check if ground treatment and a shallow foundation would be safer and/or more economical than other options such as a raft foundation or a deep foundation. Several methods for improvement of the ground are now available along with powerful equipment and rich experience of their applications.

Mechanical modification methods of ground improvement include simple compaction up to shallow depth and proportioning by triangular chart and Rothfutch methods. Deep compaction methods available namely blasting and dynamic compaction are suitable for cohesionless soils. Hydraulic modification methods include preloading, sand drains and prefabricated vertical drains address the large settlement problem of soft cohesive soils by pre-empting the settlements before construction so that the structure is safe from the danger of prohibitive post-construction settlements.

Another approach to improve the ground is by suitable reinforcement. Just as concrete when reinforced in tensile zone is able to carry more loads through bond stresses between steel and concrete, in the same way reinforcements improve the shear strength and compressive strength of the soil significantly. A wide variety of reinforcing elements and methods are available such as stone columns, reinforced earth, soil nailing and geosynthetics. All these reinforcements help to improve the shear strength and bearing capacity and reduce compression in various cases such as foundations, embankments/excavations for highways or other projects, retaining walls and earth dams. Out of the various types of reinforcing elements used, geosynthetics play multiple functions in addition to reinforcement such as filtration, drainage, separation, erosion control, sediment control and moisture barrier. Several types of geosynthetics have also come into use such as geogrids, geotextiles, geomembranes, geocomposites, each of which fulfil more than one of the previously cited functions.

Foundation grouting is a chemical stabilization method of ground improvement using several types of admixtures including cement, clay, asphalt, chemicals and grouting techniques such as compaction, fracture, permeation, slurry injection, curtain grouting and several others. *In-situ* soil mixing is an advanced technique of mixing the *in-situ* soil with cement and other materials up to large depth from ground surface using CSM, JM and DSM. Removal of groundwater helps to improve the soil properties and behavior in more than one way. Different methods of seepage control and dewatering methods are available such as well point systems, deep well systems, electro-osmosis supported by different types of pumps along with various types of cut-offs. The thermal methods of ground improvement include ground freezing and heating are powerful methods but have limited applications.

Objective Questions

1. The two basic principles of mechanical stabilization are
(a) Proportioning and compaction.
(b) Proportioning and reinforcement.
(c) Reinforcement and drainage.
(d) Drainage and dynamic compaction.

2. To achieve maximum density in mechanical stabilization as per Fuller's method, the percent finer than 4.75 mm size particles to be used in a soil having maximum particle size of 10 mm for a gradation index of 0.3 is
(a) 90%. **(c)** 70%.
(b) 80%. **(d)** 60%.

3. The materials used for mechanical stabilization in proportioning by triangular chart method are
(a) Gravel, silt, clay.
(b) Sand, silt, clay.
(c) Gravel, sand, fines.
(d) Gravel, sand, silt.

4. There are three soils which are represented graphically on a triangular chart as points A, B, and C based on % of gravel, sand and fines. If D is the point on the chart representing desirable gradation and E is the point on line AB obtained by extending the line CD, the desirable proportion of soil A, for maximum density in mechanical stabilization is given as
(a) (DE/CE) × 100.
(b) (BE/AB) × (CD/DE) × 100.
(c) (AE/AB) × (CD/DE) × 100.
(d) None of these.

5. The method of gradation that uses the entire grain-size distribution of all soils used in proportioning in mechanical stabilization is
(a) Fuller's method.
(b) Triangular chart method.
(c) Rothfutch method.
(d) All of these.

6. The principles of proportioning used in mechanical stabilization are to achieve
(a) Maximum shear strength.
(b) Minimum compression.
(c) Maximum density.
(d) Minimum permeability.

7. If d is the diameter of sand compaction piles, the diameter of the compacted zone around each pile is about
(a) $2d$ to $3d$. **(c)** $6d$ to $10d$.
(b) $4d$ to $5d$. **(d)** $7d$ to$12d$.

8. Use of sand compaction piles enables to increase the relative density of soil up to about
(a) 50%–60%. **(c)** 85%–90%.
(b) 75%–80%. **(d)** 90%–95%.

9. In ground improvement by sand compaction piles, the maximum percentage of fines and clay in the soil should be limited, respectively, to
(a) 15% and 3%. **(c)** 20% and 10%.
(b) 10% and 5%. **(d)** 25% and 10%.

10. The depth up to which soil can be improved using sand compaction piles is
(a) 5 m. **(c)** 15 m.
(b) 10 m. **(d)** 20 m.

11. To improve a soil by blasting up to depth "d," the detonators are placed at a depth from ground surface equal to
(a) $d/3$. **(c)** $3d/4$.
(b) $2d/3$. **(d)** $d/4$.

12. Detonators should be placed in more than one depth levels, if the depth of soil to be improved by blasting is more than
(a) 5 m. **(c)** 10 m.
(b) 6.5 m. **(d)** 15 m.

13. The amount of explosive having a radius of influence of about 2.9 m, that should be used to achieve densification by blasting, (when the coefficient C = 0.0025), is
(a) 10 kgf. **(c)** 14 kgf.
(b) 12 kgf. **(d)** 16 kgf.

14. In ground improvement by multi-layer blasting, the blasting should be done from
(a) top to bottom. **(c)** left to right.
(b) bottom to top. **(d)** right to left.

15. The layer of soil deposit which is most effectively improved by blasting is
(a) Surface layer. **(c)** Bottom most layer.
(b) Middle layer. **(d)** All of these.

16. The advantages of ground improvement by blasting are
   A. The method is quick.
   B. Takes less labor and cost.
   C. Useful for fine-grained soils also.
   D. There is complete control over the densification process.

   The correct answer is
   (a) A and B only. (c) C and D only.
   (b) B and C only. (d) A, B, and C only.

17. The disadvantages of ground improvement by blasting are
   A. Large equipment are required.
   B. No control over densification process.
   C. Adverse effects on adjacent structures.
   D. Un-suitable for dry or fully saturated soils.

   The correct answer is
   (a) A and B only. (c) C and D only.
   (b) B and C only. (d) A, B, and C only.

18. In ground improvement by dynamic compaction, the total weight of temper used in compaction of soils is
   (a) 10–20 kN. (c) 50–100 kN.
   (b) 20–50 kN. (d) 100–200 kN.

19. In the dynamic compaction method of ground improvement, the heavy tamping weight is made to fall from a height of
   (a) 2–5 m. (c) 10–25 m.
   (b) 5–10 m. (d) 25–50 m.

20. The number of drops of weights, the height of drop and horizontal spacing in dynamic compaction of bottom layers of soil deposit are, respectively,
   (a) Large, large, large. (c) Small, large, small.
   (b) Small, large, large. (d) Large, large, small.

21. For dynamic compaction of surface layers of a soil deposit, the number of drops, height of drop and horizontal spacing are, respectively,
   (a) Large, large, large. (c) Small, large, small.
   (b) Small, small, small. (d) Large, large, small.

22. A soil deposit is improved by dynamic compaction using a tamper of weight 15 t falling through a height of 20 m. If the empirical coefficient α is 0.5, the depth of improvement of the ground is
   (a) 12.7 m. (c) 8.7 m.
   (b) 10.7 m. (d) 6.7 m.

23. The dynamic compaction method of improving the soil is most suitable for coarse-grained soil with silt content of
   (a) <35%. (c) <15%.
   (b) <25%. (d) <5%.

24. The dynamic compaction is suitable for ground improvement of soil deposits with % clay and plasticity index, respectively, of
   (a) <35% and <15. (c) <25% and <8.
   (b) <25% and <15. (d) <15% and <8.

25. Preloading is suitable for the improvement of
   (a) Cohesionless soils. (c) Sandy gravels.
   (b) Soft cohesive soils. (d) Silty sands.

26. Sand drains improve the soils by accelerating consolidation through rapid expulsion of pore water in
   (a) Vertical direction.
   (b) Horizontal direction.
   (c) Both horizontal and vertical directions.
   (d) None of these.

27. According to Rowe, vertical drains are effective for soils with coefficient of consolidation
   (a) $<3 \times 10^{-7}$ m^2/s. (c) $<3 \times 10^{-3}$ m^2/s.
   (b) $<3 \times 10^{-5}$ m^2/s. (d) $<3 \times 10^{-2}$ m^2/s.

28. Based on equal circumstance concept, the equivalent diameter of strip drain of width B, and thickness t, is given by
   (a) $\dfrac{3(B+t)}{\pi}$. (c) $\dfrac{2(B+t)}{\pi}$.
   (b) $\dfrac{4(B+t)}{\pi}$. (d) $\dfrac{3[B+(t/2)]}{\pi}$.

29. Before installation of sand drains or strip drains, the thickness of sand layer placed over the surface of soft clay is in the range of
   (a) 0.1–0.3 m. (c) 0.5–0.7 m.
   (b) 0.3–0.5 m. (d) 0.7–0.9 m.

30. The rate of installation of prefabricated vertical drains for about 15-m long drains at the rate of 1–2.5 m c/c is about
   (a) 75 m length per hour.
   (b) 75 m length per day.
   (c) 375 m length per hour.
   (d) 375 m length per day.

31. The effect of smear due to remolding of soil during installation of prefabricated vertical drains will
   (a) Slow down radial consolidation.
   (b) Accelerate radial consolidation.
   (c) Slow down vertical consolidation.
   (d) Accelerate vertical consolidation.

32. When stone columns are subjected to ultimate load, failure takes place by bulging of
   (a) Top one-third length of column.
   (b) Middle one-third length of column.
   (c) Bottom one-third length of column.
   (d) Full length of the column.

33. For effective construction of stone columns by vibro-compaction, the maximum percent of silt should be limited to
(a) 0. **(b)** 2. **(c)** 5. **(d)** 10.

34. Vibro-compaction method of construction of stone columns is suitable for
(a) Gravels.
(b) Sands.
(c) Gravels and sands.
(d) Soft cohesive soils.

35. The method in which a vibratory probe or vibro-flot is used for construction of stone columns is known as
A. Vibro-compaction method.
B. Vibro-replacement method.
C. Vibro-composer method.
D. Cased-bore hole method.

The correct answer is
(a) A and B only. **(c)** C and D only.
(b) B and C only. **(d)** A and C only.

Review Questions

1. Briefly discuss the factors affecting the mechanical stabilization.

2. Explain how Rothfutch's graphical method is used for proportioning the materials.

3. Write a note on methods of field compaction.

4. Write a note on field compaction control.

5. Explain the factors affecting the mechanical stabilized soils.

6. Explain the criterion for selection of fill material.

7. Explain the factors affecting the cement stabilized soils.

8. Explain the design procedure of soil–cement stabilization.

9. Explain how soil–cement mix is designed using PCA and British methods.

10. Explain the types of soil cements.

11. Explain how soil–cement mix is designed using British method.

12. Explain the design procedure of soil–lime stabilization.

13. Write a note on soils amenable to lime stabilization.

14. Explain the factors affecting the lime stabilized soils.

15. Write a note on factors affecting lime stabilization.

16. Briefly discuss the factors affecting the lime–cement stabilization.

17. Write a note on factors affecting bitumen stabilization.

18. Write a note on types of soil bitumen.

19. Explain the salient futures of calcium chloride stabilization.

20. Describe the theory related to calcium chloride stabilization.

21. Briefly discuss the factors affecting the calcium chloride stabilization.

22. Write a note on sodium silicate and gypsum stabilization.

23. Briefly discuss the factors affecting the bituminous stabilization.

24. Explain the factors affecting the bitumen stabilized soils.

25. Explain the design procedure of soil–bitumen stabilization.

26. Explain how soil–bitumen mix is designed.

27. Write short notes on the following:
(a) Construction methods of soil stabilization.
(b) Field compaction.
(c) Field compaction control.
(d) Lime–cement stabilization.
(e) Lime–fly ash stabilization.
(f) Gypsum stabilization.
(g) Sodium silicate stabilization.
(h) Calcium chloride stabilization.
(i) Calcium chloride–sodium silicate stabilization.

28. Write a note on method of blasting for densification of cohesionless soils.

29. Explain the vibration at the ground surface.

30. Write a note on vibration technique for densification of cohesion less soils.

31. Write short notes on the following: Impact at depth.

32. Write a note on *in-situ* densification methods in cohesion less soils.
33. Explain the pre-loading technique with the help of neat sketch.
34. Explain how pre-loading technique is useful in improving the properties of the soil.
35. Write a note on sand wicks.
36. Explain how sand drains are effective in improving the properties of the soil.
37. Explain the installations techniques of sand drains with the help of neat sketch.
38. Explain how sand wicks are effective in the stabilization of soils.
39. Write short notes on the following: Blanket drains.
40. Explain the soil replacement technique used for improving the problematic soils.
41. Explain how the stone columns are useful for improving the properties of soil.
42. Explain the installations techniques of stone column with the help of neat sketch.
43. Explain how the stone columns are installed using ramming technique.
44. Explain how the stone columns are installed using vibro-flotation technique.
45. Write a note on vibro-compaction technique for densification of cohesion less soils.
46. Explain how the stone columns are installed by Ramming technique with the help of neat sketch.
47. Discuss the differences between sand drains and stone columns.
48. Discuss the components of the reinforced earth wall with the help of neat sketch.
49. Discuss the principles of the reinforced earth wall.
50. Discuss the design steps of the reinforced earth wall with the help of neat sketch.
51. Write short notes on the following:
    (a) Components of reinforced earth.
    (b) Components of reinforced earth walls.
    (c) Design principles of reinforced earth walls.
    (d) Principles of reinforced earth.
52. Write a note on various functions of geosynthetics with the help of neat sketches.
53. Write a note on various applications of geosynthetics.
54. Briefly discuss the functions and applications of geosynthetics.
55. Write a note on various properties of geosynthetics with the help of neat sketches.
56. Write a note on properties of geotextile.
57. Discuss in brief about the various testing methods for geotextile materials.
58. Discuss in brief about the various properties and testing methods for geotextile materials.
59. Write a note on geodrains.
60. Explain the concept of geodrains with the help of sketch.
61. Write short notes on the following:
    (a) Functions of geosynthetics.
    (b) Properties of geotextiles.
    (c) Geodrains.
62. Write a note on objects of grouting.
63. What are the different types of grouting? Explain?
64. Write a note on grouting methods.
65. Describe the theory related to lime slurry pressure injection technique along with the applications.
66. Write a note on different applications of grouting with the help of neat sketches.
67. Write a note on stage grouting.
68. Write a note on post-grout test.
69. Write a note on vacuum dewatering.
70. Explain the criterion for selection of fill material around drains.
71. Write a note on deep well system of dewatering method.
72. Write a note on deep well drainage technique for densification of soils.
73. Write a note on multi-stage well point system of dewatering technique.
74. Write a note on vacuum dewatering technique for densification of cohesive soils.
75. Explain how electro-osmosis technique is effective for dewatering in cohesive soils.
76. Write short notes on the following:
    (a) Vacuum dewatering.
    (b) Hydraulic functioning in soil.
77. Write a note on thermal stabilization.
78. Write a note on thermal stabilization by heating.
79. Write a note on stabilization by cooling.

Answers

Objective Questions

1. (a)
2. (b)
3. (c)
4. (b)
5. (c)
6. (c)
7. (d)
8. (b)
9. (a)
10. (d)
11. (b)
12. (c)
13. (a)
14. (b)
15. (c)
16. (a)
17. (b)
18. (d)
19. (c)
20. (a)
21. (b)
22. (c)
23. (a)
24. (c)
25. (b)
26. (b)
27. (a)
28. (c)
29. (b)
30. (b)
31. (a)
32. (a)
33. (d)
34. (c)
35. (a)