PART TWO

Building Construction

10

Foundations

10–1 FOUNDATION SYSTEMS

The *foundation* of a structure supports the weight of the structure and its applied loads. In a broad sense, the term "foundation" includes the soil or rock upon which a structure rests, as well as the structural system designed to transmit building loads to the supporting soil or rock. Hence the term *foundation failure* usually refers to collapse or excessive settlement of a building's supporting structure resulting from soil movement or consolidation rather than from a failure of the foundation structure itself. In this chapter the term "foundation" will be used in its more limited sense to designate those structural components that transfer loads to the supporting soil or rock.

A foundation is a part of a building's substructure—that portion of the building which is located below the surrounding ground surface. The principal types of foundation systems include spread footings, piles, and piers. These are illustrated in Figure 10–1 and described in the following sections.

One method of describing a building's construction is based on the location of the lowest building floor. In this method, the types of construction include slab-on-grade construction, crawl space construction, and basement construction. In *slab-on-grade* construction, the lowest floor of the building rests directly on the ground. In *crawl space* construction, the lowest floor of the building is suspended a short distance (less than a full floor height) above the ground. The crawl space provides convenient access to utility lines and simplifies the installation of below-the-floor utilities. *Basement* construction provides one or more full stories below ground level. The use of basements provides storage space or additional living space at relatively low cost. Unless carefully constructed, however, basements are often troubled by water leakage or dampness.

10–2 SPREAD FOOTINGS

A *spread footing* is the simplest and probably the most common type of building foundation. It usually consists of a square or rectangular reinforced concrete pad that serves to distribute building loads over an area large enough so that the resulting pressure on the supporting soil

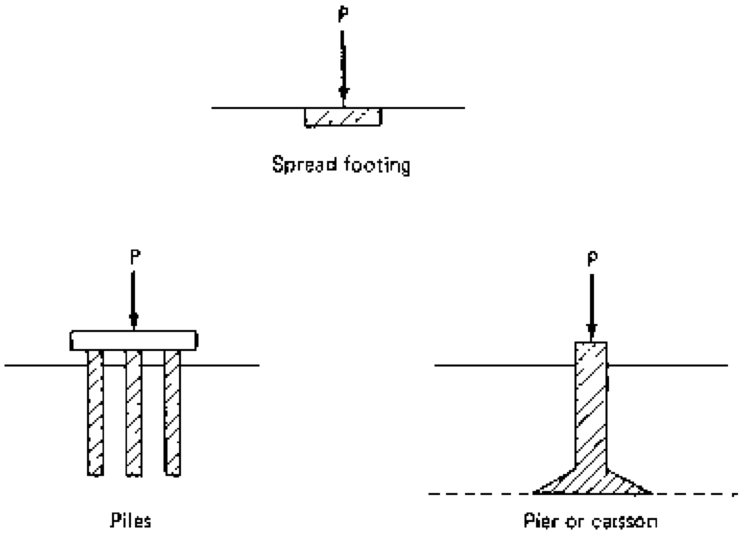


Figure 10–1 Foundation systems.

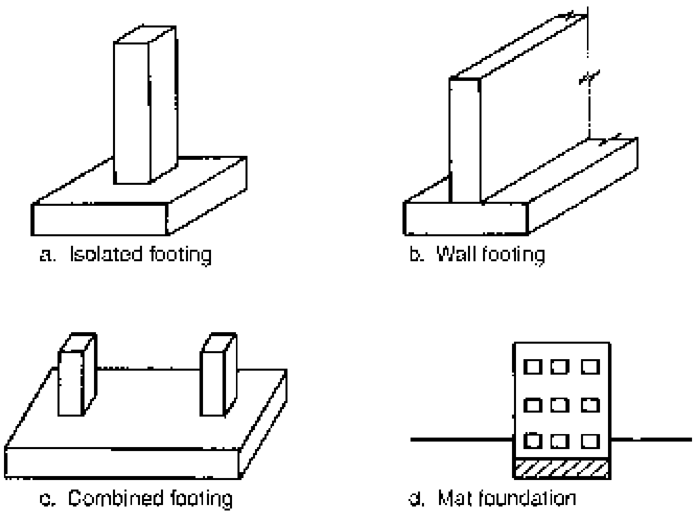


Figure 10–2 Types of spread footings.

does not exceed the soil's allowable bearing strength. The principal types of spread footings are illustrated in Figure 10–2. They include individual footings, combined footings, and mat foundations. *Individual footings* include isolated (or single) footings, which support a single column (Figure 10–2a), and wall footings (Figure 10–2b), which support a wall. *Combined footings* support a wall and one or more columns, or several columns (Figure 10–2c).

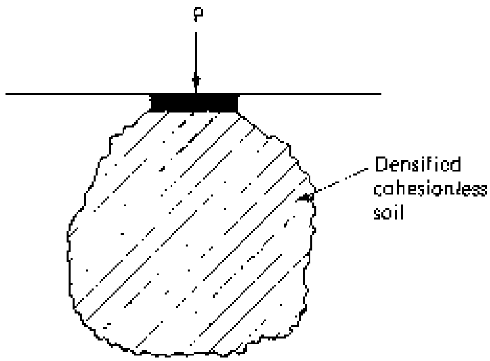


Figure 10–3 Soil densification under footing.

Mat or *raft foundations* (Figure 10–2d) consist of a heavily reinforced concrete slab extending under the entire structure, in order to spread the structure's load over a large area. Because such foundations are usually employed for large buildings, they generally involve deep excavation and large-scale concrete pours. A *floating foundation* is a type of mat foundation in which the weight of the soil excavated approximately equals the weight of the structure being erected. Thus, in theory, the erection of the building would not result in any change in the load applied to the soil and hence there would be no settlement of the structure. In practice, however, some soil movement does occur, because the soil swells (or rebounds) during excavation and then recompresses as the building is erected.

If the underlying soil can be strengthened, the allowable bearing pressure on the soil surface will be increased. As a result, it may be possible to use spread footings for foundation loads that normally would require piles or other deep foundation methods. The process of improving soils in place, called *ground modification* or *soil stabilization,* is described in Section 5–3. In addition to improving bearing capacity, ground modification may also reduce foundation settlement, groundwater flow, and the subsidence resulting from seismic action. An example of soil densification resulting from vibratory compaction under a footing is illustrated in Figure 10–3.

10–3 PILES

A *pile* is nothing more than a column driven into the soil to support a structure by transferring building loads to a deeper and stronger layer of soil or rock. Piles may be classified as either end-bearing or friction piles, according to the manner in which the pile loads are resisted. However, in actual practice, virtually all piles are supported by a combination of skin friction and end bearing.

Pile Types

The principal types of piles include timber, precast concrete, cast-in-place concrete, steel, composite, and bulb piles. *Timber piles* are inexpensive, easy to cut and splice, and require no special handling. However, maximum pile length is limited to about 100 ft, load-carrying ability is limited, and pile ends may splinter under driving loads. Timber piles are also subject to insect attack and decay. However, the availability of pressure-treated wood described in Chapter 13 has greatly reduced the vulnerability of timber piles to such damage.

Precast concrete piles may be manufactured in almost any desired size or shape. Commonly used section shapes include round, square, and octagonal shapes. Advantages of concrete piles include high strength and resistance to decay. However, a precast concrete pile is usually the heaviest type of pile available for a given pile size. Because of their brittleness and lack of tensile strength, they require care in handling and driving to prevent pile damage. Since they have little strength in bending, they may be broken by improper lifting procedures. Cutting requires the use of pneumatic hammers and cutting torches or special saws. Splicing is relatively difficult and requires the use of special cements.

Cast-in-place concrete piles (or shell piles) are constructed by driving a steel shell into the ground and then filling it with concrete. Usually, a steel mandrel or core attached to the pile driver is placed inside the shell to reduce shell damage during driving. Although straight shells may be pulled as they are filled with concrete, shells are usually left in place and serve as additional reinforcement for the concrete. The principal types of shell pile include uniform taper, step-taper, and straight (or monotube) piles. The shells for cast-in-place piles are light, easy to handle, and easy to cut and splice. Since shells may be damaged during driving, they should be visually inspected before filling with concrete. Shells driven into expansive soils should be filled with concrete as soon as possible after driving to reduce the possibility of shell damage due to lateral soil pressure.

Steel piles are capable of supporting heavy loads, can be driven to great depth without damage, and are easily cut and spliced. Common types of steel piles include H-piles and pipe piles, where the name indicates the shape of the pile section. Pipe piles are usually filled with concrete after driving to obtain additional strength. The principal disadvantage of steel pile is its high cost.

Composite piles are piles made up of two or more different materials. For example, the lower section of pile might be timber while the upper section might be a shell pile. This would be an economical pile for use where the lower section would be continuously submerged (hence not subject to decay) while the upper section would be exposed to decay.

Bulb piles are also known as *compacted concrete piles, Franki piles,* and *pressure-injected footings.* They are a special form of cast-in-place concrete pile in which an enlarged base (or bulb) is formed during driving. The enlarged base increases the effectiveness of the pile as an end bearing pile. The driving procedure is illustrated in Figure 10–4. A drive tube is first driven to the desired depth of the base either by a powered hammer operating on the top of the drive tube (called *top driving*) or by placing a plug of zero-slump concrete [concrete having a slump of 1 in. (25 mm) or less] into the drive tube and driving both the concrete plug and the drive tube simultaneously using a drop hammer operating inside the drive tube (called *bottom driving*). The drive tube is then held in place and more zero-slump concrete added and hammered out of the end of the drive tube to form the base. Finally, the body or shaft of the pile is constructed by either of two methods. A compacted concrete shaft is formed by hammering zero-slump concrete into the ground as the drive tube is raised. A cased shaft is constructed by placing a steel shell inside the drive tube and then hammering a plug of zero-slump concrete into place to form a bond between the base and the shell. The shell is then filled in the same manner as a conventional cast-in-place

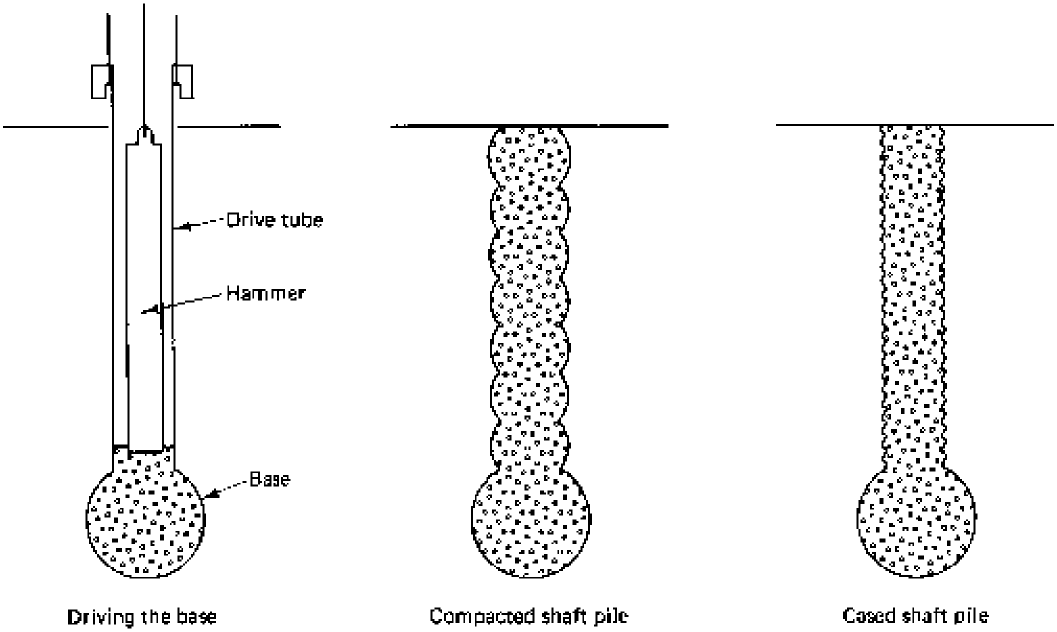


Figure 10–4 Bulb piles.

concrete pile. Compacted shaft piles usually have a higher load capacity than do cased shaft piles due to the increased pressure between the shaft and the surrounding soil.

Minipiles or *micro piles* are small-diameter [2 to 8 in. (5 to 20 cm)], high-capacity [to 60 tons (54 t)] piles. They are most often employed in areas with restricted access or limited headroom to underpin (provide temporary or additional support to) building foundations. Some other applications include strengthening bridge piers and abutments, anchoring or supporting retaining walls, and stabilizing slopes. While they may be driven in place, minipiles are often installed by drilling a steelcased hole 2 to 8 in. (5 to 20 cm) in diameter, placing reinforcing in the casing, and then bonding the soil, casing, and reinforcement together by grouting.

Pile Driving

In ancient times, piles were driven by raising and dropping a weight such as a large stone onto the pile. The drop hammer, the modern version of this type of pile driver, is illustrated in Figure 10–5. As you see, the pile-driving assembly is attached to a mobile crane, which provides the support and the power for the pile driver. The *leads* act as guides for the drop weight and the pile. Driving operations consist of lifting the pile, placing it into the leads, lowering the pile until it no longer penetrates the soil under its own weight, and then operating the drop hammer until the pile is driven to the required resistance. Safety requirements for drop hammers include the use of stop blocks to prevent the hammer from being raised against the head block (which could result in collapse of the boom), the use of a guard

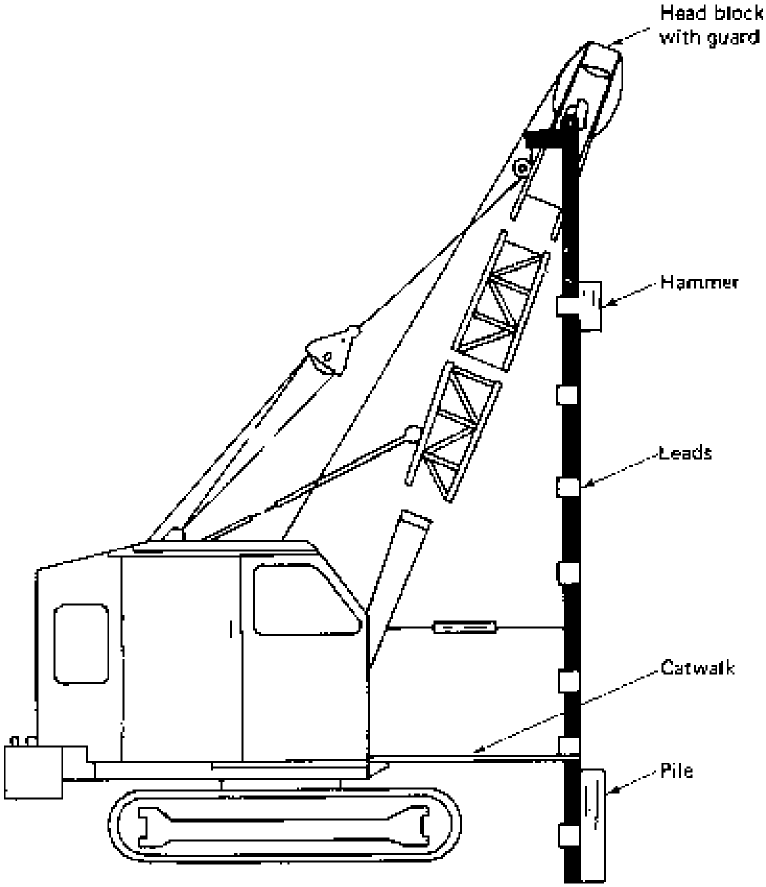


Figure 10–5 Drop hammer pile driver.

across the head block to prevent the drop cable from jumping out of the sheaves, and placing a blocking device under the hammer whenever workers are under the hammer.

The remaining types of pile drivers are all powered hammers. That is, they use a working fluid rather than a cable to propel the ram (driving weight). Early powered hammers used steam as a working fluid. Steam power has now been largely replaced by compressed air power. Hydraulic power is replacing compressed air in many newer units. Single-acting hammers use fluid power to lift the ram, which then falls under the force of gravity. Double-acting and differential hammers use fluid power to both lift the ram and then drive the ram down against the pile. Thus double-acting and differential hammers can be lighter than a single-acting hammer of equal capacity. Typical operating frequencies are about 60 blows/min for single-acting hammers and 120 blows/min for double-acting hammers. Differential hammers usually operate at frequencies between these two values.

A diesel hammer contains a free-floating ram-piston that operates in a manner similar to that of a one-cylinder diesel engine. The principle of operation is illustrated in Figure 10–6.

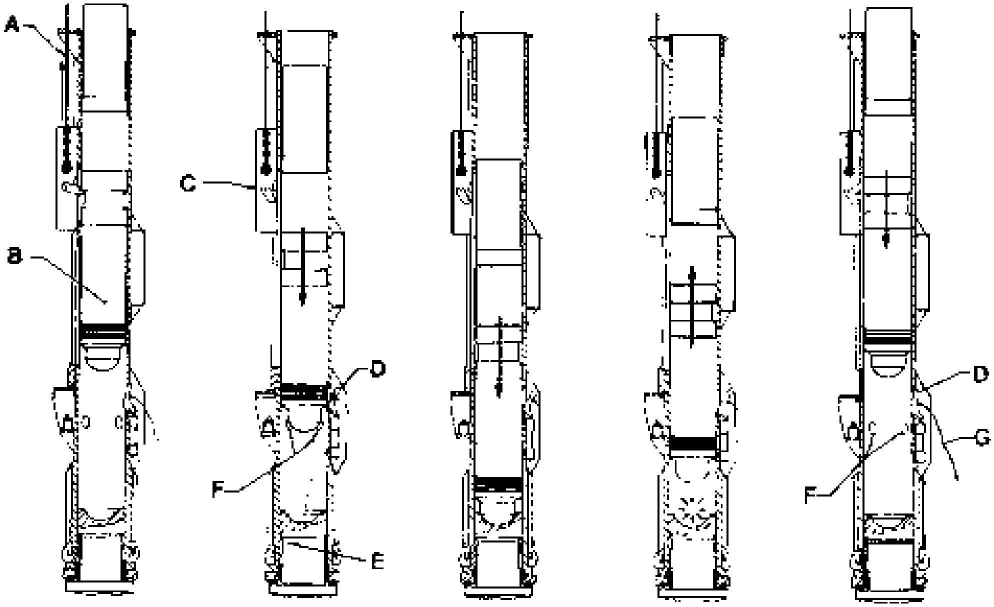


Figure 10–6 Operation of a diesel pile hammer. (Courtesy of MKT Manufacturing, Inc.)

The hammer is started by lifting the ram (B) with the crane hoist line (A). The trip mechanism (C) automatically releases the ram at the top of the cylinder. As the ram falls, it actuates the fuel pump cam (D), causing fuel to be injected into the fuel cup in the anvil (E) at the bottom of the cylinder. As the ram continues to fall, it blocks the exhaust ports (F), compressing the fuel-air mixture. When the ram strikes the anvil, it imparts an impact blow to the pile top and also fires the fuel-air mixture. As the cylinder fires, it forces the body of the hammer down against the pile top and drives the ram upward to start a new cycle. Operation of the hammer is stopped by pulling the rope (G), which disengages the fuel pump cam (D). Diesel hammers are compact, light, and economical and can operate in freezing weather. However, they may fail to operate in soft soil, where hammer impact may be too weak to fire the fuel-air mixture.

Vibratory hammers drive piles by a combination of vibration and static weight. As you might expect, they are most effective in driving piles into clean granular soils. Sonic hammers are vibratory hammers that operate at very high frequencies. Figure 10–7 shows a hydraulically powered vibratory driver/extractor in operation.

Pile-Driving Procedures

A typical pile-driving operation for a straight shell pile is illustrated in Figure 10–8. Figure 10–8a shows the piles stockpiled at the job site. Notice the depth marks that have been painted on the pile. These will be used during driving to facilitate counting the number of blows required to obtain a foot of penetration. In Figure 10–8b, the pile has been hooked to the hoist cable and is being swung into position for lowering into a hollow casing previously driven into the ground. After the shell has been lowered into the casing, the pile driver's mandrel (Figure 10–8c) is

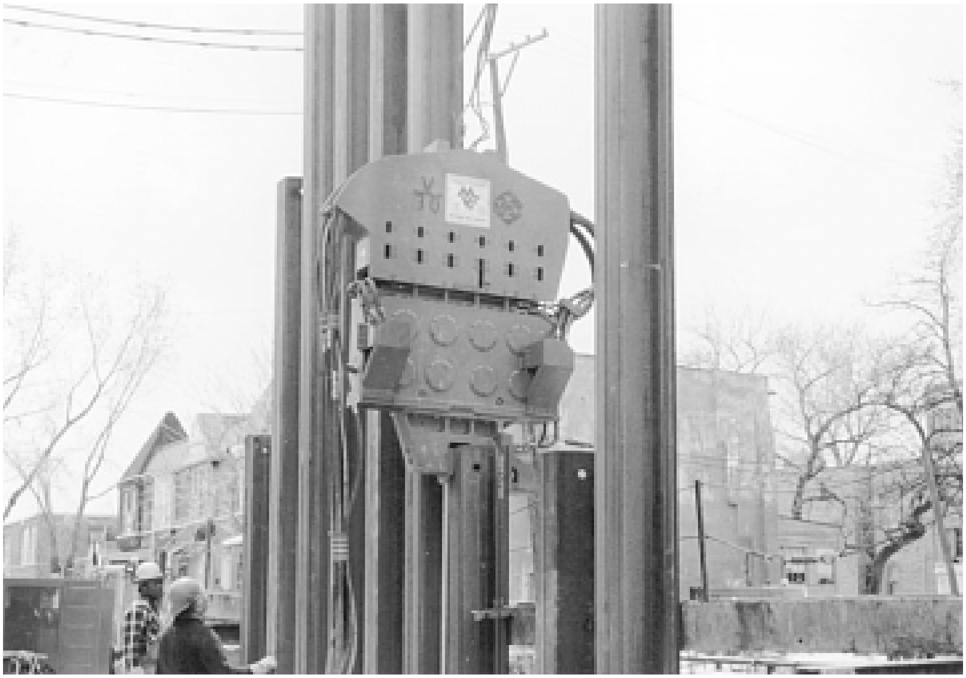


Figure 10–7 Hydraulically powered vibratory driver/extractor. (Courtesy of MKT Manufacturing, Inc.)

lowered into the shell. The shell and mandrel are then raised from the casing and swung into position for driving. The hammer (in this case a single-acting compressed air hammer) then drives the pile (Figure 10–8d) until the required depth or driving resistance is obtained. After the mandrel is raised, the shell is cut off at the required elevation with a cutting torch (Figure 10–8e). When the reinforcing steel for the pile cap has been placed (Figure 10–8f), the shell is ready to be filled with concrete.

For driving piles with an impact-type pile driver it is recommended that a hammer be selected that will yield the required driving resistance at a final penetration of 8 to 12 blows/in. (reference 6). For fluid-powered hammers, it is also recommended that the weight of the ram be at least one-half the pile weight. For diesel hammers, ram weight should be at least one-fourth of the pile weight. When selecting a vibratory driver/extractor, a machine should be used that will yield a driving amplitude of $\frac{1}{4}$ to $\frac{1}{2}$-in. (0.6 to 1.2 cm).

$$\text{Driving amplitude (in.)} = 2 \times \frac{\text{Eccentric moment (in.-lb)}}{\text{Vibrating mass (lb)}} \qquad \textbf{(10–1)}$$

In solving Equation 10–1 for driving amplitude, pile weight should be added to the weight of the driver's vibrating mass to obtain the value of the vibrating mass.

Powered hammers with leads should be used for driving piles at an angle (batter piles), because drop hammers lose significant energy to friction when the leads are inclined.

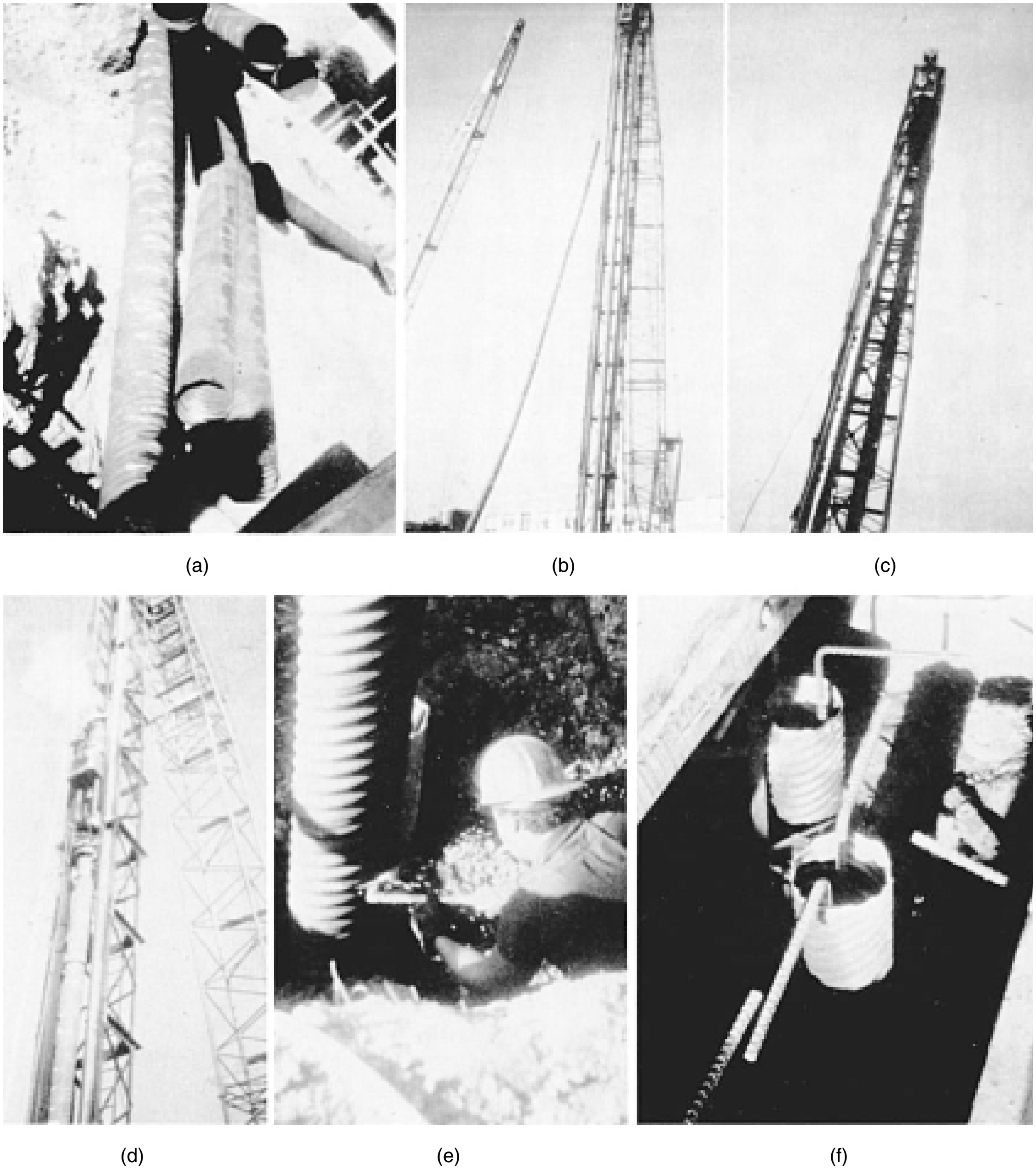


Figure 10–8 Driving a shell pile.

Powered hammers without leads may be used in vertical driving, but the use of leads assists in maintaining pile alignment during driving. Double-acting, differential, and vibratory hammers may be used to extract piles as well as to drive them.

Determining Pile Load Capacity

The problem of determining pile load capacity is a complex one since it involves pile–soil–hammer interaction during driving, pile–soil interaction after the pile is in place, and the structural strength of the pile itself. The geotechnical engineer who designs the foundation must provide a pile design that is adequate to withstand driving stresses as well as to support the design load of the structure without excessive settlement. The best measure of in-place pile capacity is obtained by performing pile load tests as described later in this section.

A number of dynamic driving equations have been developed in attempting to predict the safe load capacity of piles based on behavior during driving. The traditional basis for such equations is to equate resisting energy to driving energy with adjustments for energy lost during driving. These equations treat the pile as a rigid body. A number of modifications to basic driving equations have been proposed in an attempt to provide better agreement with measured pile capacity. Equation 10–2, for determining the safe capacity of piles driven by powered hammers, has been incorporated in some U.S. building codes. Minimum hammer energy may also be specified by the building code.

$$R = \left(\frac{2E}{S + 0.1}\right)\left(\frac{W_r + KW_p}{W_r + W_p}\right) \tag{10–2}$$

where R = safe load (lb)

S = average penetration per blow, last six blows (in.)

E = energy of hammer (ft-lb)

K = coefficient of restitution $\begin{cases} 0.2 \text{ for piles weighing 50 lb/ft or less} \\ 0.4 \text{ for piles weighing 50 to 100 lb/ft} \\ 0.6 \text{ for piles weighing over 100 lb/ft} \end{cases}$

W_r = weight of hammer ram (lb)

W_p = weight of pile, including driving appurtenances (lb)

EXAMPLE 10–1

Using Equation 10–2 and the driving data below, determine the safe load capacity of a 6-in.-square concrete pile 60 ft long. Assume that the unit weight of the pile is 150 lb/cu ft.

Pile driver energy = 14,000 ft-lb

Ram weight = 4000 lb

Weight of driving appurtenances = 1000 lb

Average penetration last six blows = 1/5 in./blow

SOLUTION

$$\text{Weight of pile} = \frac{6 \times 6}{144} \times 60 \times 150 - 2250 \text{ lb}$$
$$W_p = 2250 + 1000 = 3250 \text{ lb}$$
$$\text{Weight per foot of pile} = \frac{2250}{60} = 37.5 \text{ lb/ft}$$
$$K = 0.2$$
$$S = 0.2 \text{ in./blow}$$
$$R = \left(\frac{2E}{S + 0.1}\right)\left(\frac{W_r + KW_p}{W_r + W_p}\right)$$
$$= \left(\frac{(2)(14{,}000)}{0.2 + 0.1}\right)\left(\frac{4000 + (0.2)(3250)}{4000 + 3250}\right)$$
$$= \frac{(28{,}000)(4650)}{(0.3)(7250)} = 59{,}862 \text{ lb}$$

Equation 10–3 is used in several building codes and construction agency specifications for predicting the safe load capacity of bulb piles.

$$L = \frac{W \times H \times B \times V^{2/3}}{K} \qquad \textbf{(10–3)}$$

where L = safe load capacity (tons)
W = weight of hammer (tons)
H = height of drop (ft)
B = number of blows per cubic foot of concrete used in driving final batch into base
V = uncompacted volume of concrete in base and plug (cu ft)
K = dimensionless constant depending on soil type and type of pile shaft

Nordlund (reference 11) has presented recommended K values which range from 9 for a compacted shaft pile in gravel to 40 for a cased shaft pile in very fine sand.

EXAMPLE 10–2

Calculate the safe load capacity of a bulb pile based on the following driving data.

Hammer weight = 3 tons
Height of drop = 20 ft
Volume in last batch driven = 5 cu ft
Number of blows to drive last batch = 40
Volume of base and plug = 25 cu ft
Selected K value = 25

SOLUTION

$$B = \frac{40}{5} = 8 \text{ blows/cu ft}$$

$$R = \frac{W \times H \times B \times V^{2/3}}{K}$$

$$= \frac{(3)(20)(8)(25)^{2/3}}{25} = 164 \text{ tons}$$

A newer and better approach to predicting pile capacity is provided by the use of wave equation analysis which analyzes the force and velocity waves developed in a pile as a result of driving. For pile design, the analysis is based on a specific type and length of pile, a specific driving system, and the expected soil conditions. Computer wave equation analysis programs, such as the WEAP (Wave Equation Analysis of Pile Driving) program, are available for analyzing wave data to predict pile behavior during driving and to confirm pile performance during construction. Pile driving analyzers which measure and analyze the force and velocity waves actually developed during driving are also available. They are particularly useful for establishing pile driving criteria and for quality control during driving. They can be used to measure hammer efficiency, driving energy delivered to the pile, and to indicate pile breakage during driving. The pile driving analyzer may be successfully employed to predict the capacity of production piles when results are correlated with pile load tests and good driving records.

Pile capacity may be determined by performing pile load tests. One such test procedure (ASTM D-1143) involves loading the pile to 200% of design load at increments of 25% of the design load. Each load increment is maintained until the rate of settlement is not greater than 0.01 in./h (0.25 mm/h) or until 2 h have elapsed. The final load (200% of design load) is maintained for 24 h. Quick load tests utilizing a constant rate of penetration test or a maintained load test are also used. Quick load tests can usually be performed in 3 h or less.

With all of the methods of load testing, pile settlement is plotted against load to determine pile capacity. A number of methods have been proposed for identifying the failure load on a load-settlement curve (references 7 and 8). One procedure for interpreting the load-settlement curve to determine ultimate pile capacity, often called the tangent method, involves drawing tangents to the initial and final segments of the curve as illustrated in Figure 10–9. The load (A) corresponding to the intersection of these two tangents is designated the ultimate pile capacity. A procedure used by the U.S. Army Corps of Engineers determines ultimate pile capacity as the average of the following three values:

1. A settlement of 0.25 in. (6.35 mm) on the net settlement vs. load curve.
2. The load determined by the tangent method previously described.
3. The load that corresponds to the point on the net settlement vs. load curve where the slope equals 0.01 in. per ton (0.28 mm/t).

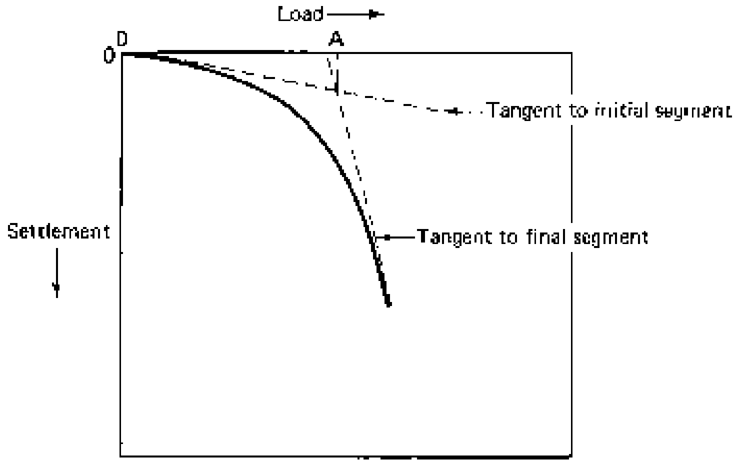


Figure 10–9 Determination of pile capacity from load test.

To determine safe pile load capacity, an appropriate factor of safety must be applied to the ultimate pile capacity. The factor of safety to be used will depend on soil–pile properties and the loading conditions to be encountered. However, a minimum factor of safety of 2.0 is usually employed.

Pile capacity usually increases after a period of time following driving. This increase in capacity is referred to as *soil setup* or *soil freeze.* However, in some cases pile capacity decreases with time. This decrease in capacity is referred to as *soil relaxation.* Soil setup or soil relaxation can be measured by performing load tests or by restriking the pile several days after pile driving. Building codes may specify a minimum waiting period between driving and loading a test pile.

10–4 PIERS AND CAISSONS

A *pier* is simply a column, usually of reinforced concrete, constructed below the ground surface. It performs much the same function as a pile. That is, it transfers the load of a structure down to a stronger rock or soil layer. Piers may be constructed in an open excavation, a lined excavation (caisson), or a drilled excavation. Since piers are often constructed by filling a caisson with concrete, the terms pier foundation, caisson foundation, and drilled pier foundation are often used interchangeably.

A *caisson* is a structure used to provide all-around lateral support to an excavation. Caissons may be either open or pneumatic. Pneumatic caissons are air- and watertight structures open on the bottom to permit the excavation of soil beneath the caisson. The caisson is filled with air under pressure to prevent water and soil from flowing in as excavation proceeds. To prevent workers from suffering from the bends upon leaving pneumatic caissons, they must go through a decompression procedure like that employed for divers. Because of the health hazards and expense of this procedure, pneumatic caissons are rarely used today.

Drilled piers are piers placed in holes drilled into the soil. Holes drilled into cohesive soils are not usually lined. If necessary, the holes may be filled with a slurry of clay and water (such as bentonite slurry) during drilling to prevent caving of the sides. Concrete is then placed in the hole through a tremie, displacing the slurry. This procedure is similar to the slurry trench excavation method described in Section 10–6.

Holes drilled in cohesionless soils must be lined to prevent caving. Metal or fiber tubes are commonly used as liners. Linings may be left in place or they may be pulled as the concrete is placed. Holes for drilled piers placed in cohesive soil are often widened (or belled) at the bottom, as shown in Figure 10–1, to increase the bearing area of the pier on the supporting soil. Although this increases allowable pier load, such holes are more difficult to drill, inspect, and properly fill with concrete than are straight pier holes.

10–5 STABILITY OF EXCAVATIONS

Slope Stability

To understand the principal modes of slope failure, it is necessary to understand the basic concepts of soil strength. The soil identification procedures discussed in Chapter 2 included the classification of soil into cohesionless and cohesive types. As you recall, cohesionless soil is one whose grains do not show any tendency to stick together. The shear strength of a cohesionless soil is thus due solely to the friction developed between soil grains. A normal force (or force perpendicular to the sliding surface) is required to develop this strength. When an embankment composed of a cohesionless soil fails, it fails as shown in Figure 10–10. That is, material from the upper part of the slope breaks away and falls to the toe of the slope until the face of the embankment reaches the natural angle of repose for the soil.

In a cohesive soil, on the other hand, shear strength is provided primarily by the attraction between soil grains (which we call *cohesion*). Theoretically, a completely cohesive soil would exhibit no friction between soil grains. Failure of a highly cohesive soil typically occurs as shown in Figure 10–11. Notice that a large mass of soil has moved along a surface,

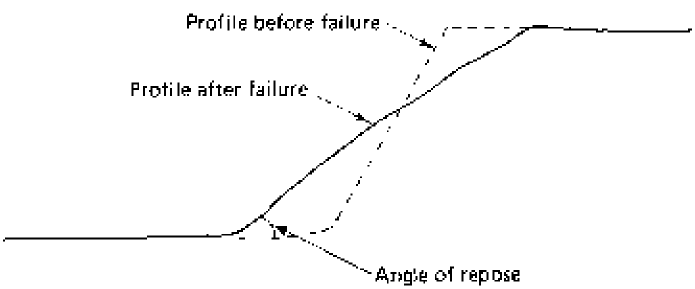


Figure 10–10 Slope failure of cohesionless soil.

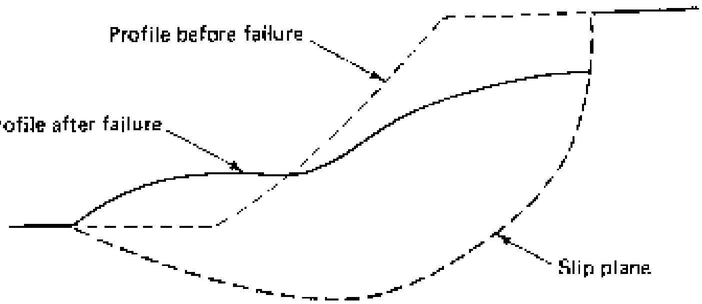


Figure 10–11 Slope failure of cohesive soil.

which we call a *slip plane.* The natural shape of this failure surface resembles the arc of an ellipse but is usually considered to be circular in soil stability analyses.

Embankment Failure During Construction

Most soils encountered in construction exhibit a combination of the two soil extremes just described. That is, their shear strength is from a combination of intergranular friction and cohesion. However, the behavior of a highly plastic clay will closely approximate that of a completely cohesive soil.

Theoretically, a vertical excavation in a cohesive soil can be safely made to a depth that is a function of the soil's cohesive strength and its angle of internal friction. This depth can range from under 5 ft (1.5 m) for a soft clay to 18 ft (5.5 m) or so for a medium clay. The safe depth is actually less for a stiff clay than for a medium clay, because stiff clays commonly contain weakening cracks or fissures. In practice, however, the theoretically safe depth of unsupported excavation in clay can be sustained for only a limited time. As the clay is excavated, the weight of the soil on the sides of the cut causes the sides of the cut to bulge (or move inward at the bottom) with an accompanying settlement (or subsidence) of the soil at the top of the cut, as shown in Figure 10–12. Subsidence of the soil at the top of the cut usually results in the formation of tension cracks on the ground surface, as shown in Figure 10–13. Such cracks usually occur at a distance from the face of the cut equal to 1/2 to 2/3 of the depth of the cut. If lateral support is not provided, tension cracks will continue to deepen until failure of the embankment occurs. Failure may occur by sliding of the soil face into the cut (Figure 10–14a) or by toppling of the upper part of the face into the cut (Figure 10–14b).

The stability of an embankment or excavation is also affected by external factors. These include weather conditions, ground water level, the presence of loads such as material and equipment near the top of the embankment/excavation, and the presence of vibration from equipment or other sources (see also Section 19–4).

Stability of Cut Bottom

Whenever cohesive soil is excavated, heaving (or rising) of the bottom of the cut will occur due to the weight of the soil on the sides of the cut. Heaving is most noticeable when

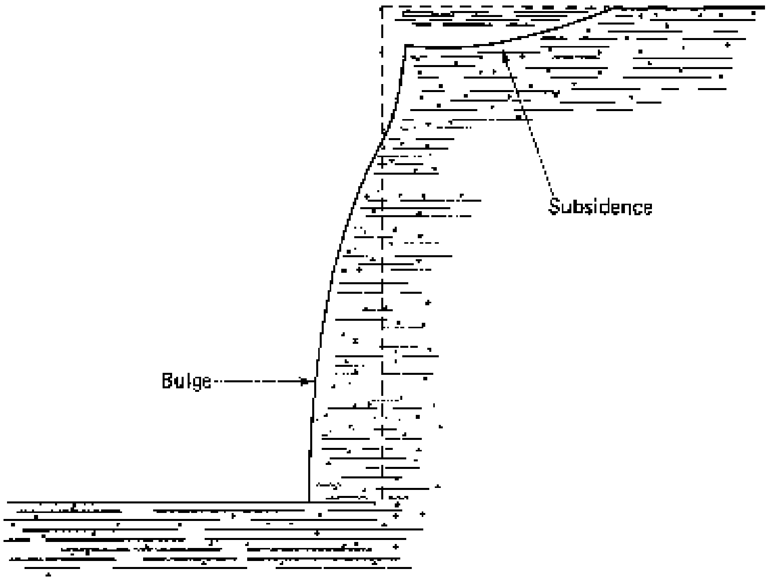


Figure 10–12 Subsidence and bulging.

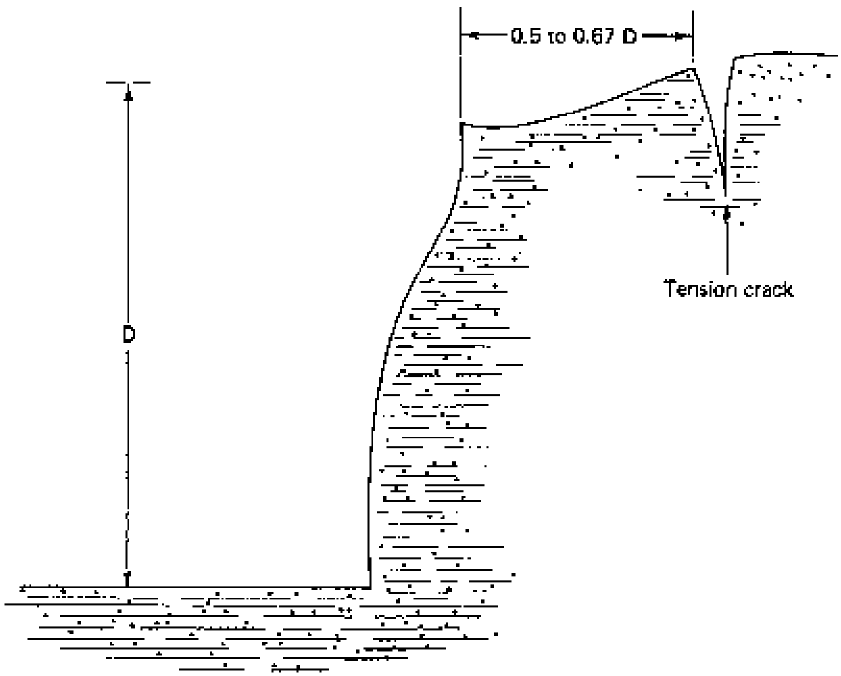


Figure 10–13 Formation of tension crack.

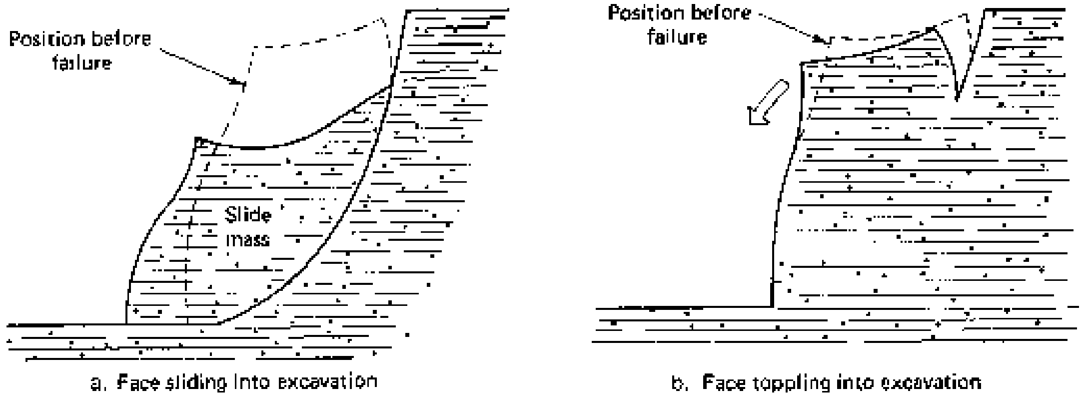


Figure 10–14 Modes of embankment failure.

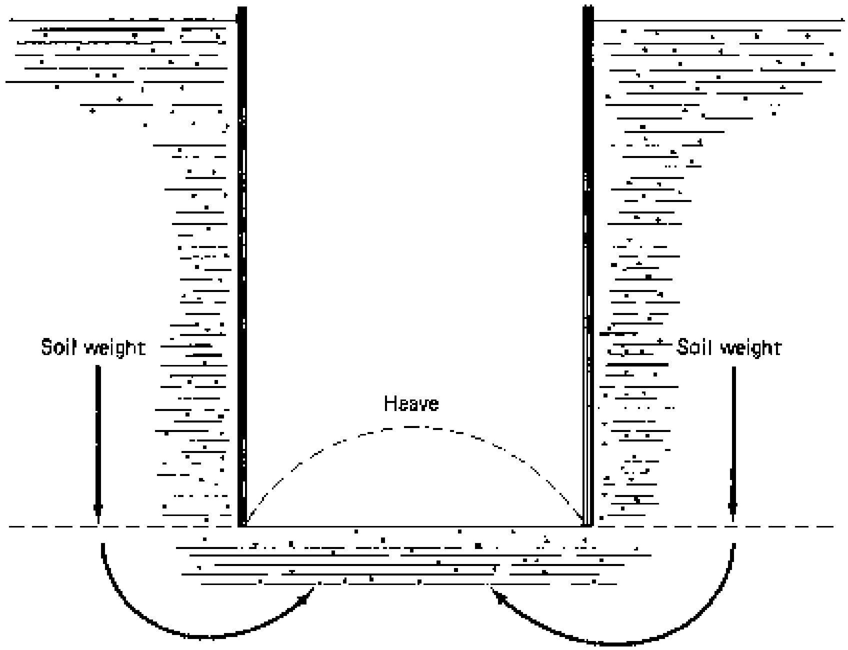


Figure 10–15 Heaving of cut bottom.

the sides of the cut have been restrained, as shown in Figure 10–15. A more serious case of bottom instability may occur in cohesionless soils when a supply of water is present. If the sides of the cut are restrained and the bottom of the cut is below the groundwater level, water will flow up through the bottom of the excavation, as shown in Figure 10–16. The upward flow of water reduces the effective pressure between the soil grains in the bottom of

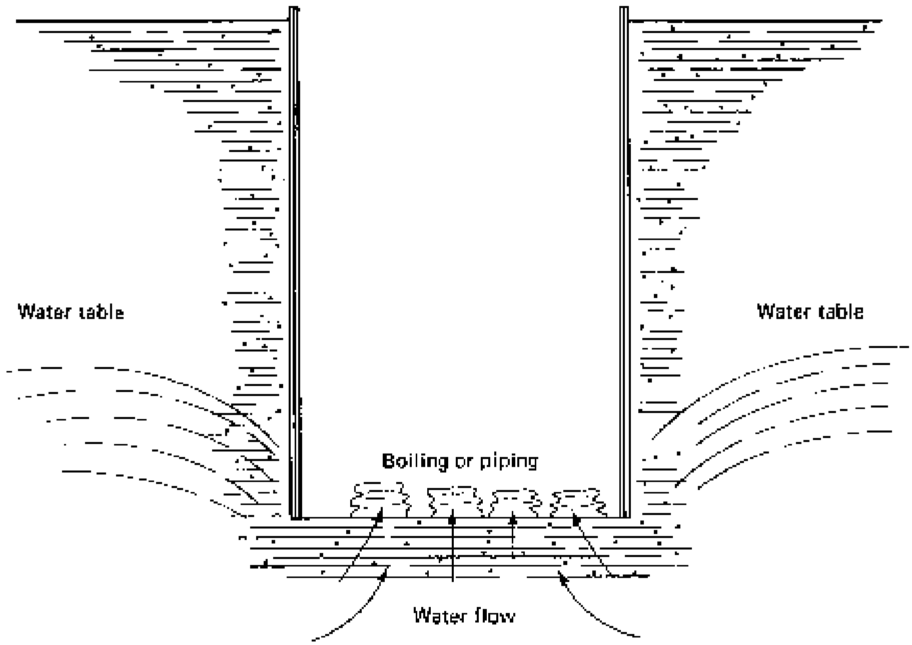


Figure 10–16 Boiling and piping of cut bottom.

the cut. This may result in one of several different conditions. If the water pressure exactly equals soil weight, the soil will behave like a liquid and we have a condition called *liquefaction* (or quicksand). Such a soil is unable to support any applied load. If the water pressure is strong enough to move subsurface soil up through the bottom of the cut, this condition is called *boiling* or *piping.* Such a movement of soil often leads to failure of the surrounding soil. This has been the cause of the failure of some dams and levees.

Preventing Embankment Failure

An analysis of the causes of excavation slope failure described above will indicate methods that can be used to prevent such failures. Side slopes may be stabilized by cutting them back to an angle equal to or less than the angle of repose of the soil, or by providing lateral support for the excavation as discussed in Section 10–6. Both side and bottom stability may be increased by dewatering the soil surrounding the excavation. Methods for dewatering and protecting excavations are described in the following sections.

To protect more permanent slopes, such as highway cuts, retaining walls are often used. Slopes of cohesive soil may be strengthened by increasing the shearing resistance along the potential slip plane. This may be done by driving piles or inserting stone columns into the soil across the potential slip plane. Another technique for reinforcing slopes is called *soil* (or earth) *reinforcement.* One form of this process is known under the trademark name Reinforced Earth. As shown in Figure 10–17, soil reinforcement involves embedding high-tensile-strength nonbiodegradable elements in a compacted soil mass. The embedded

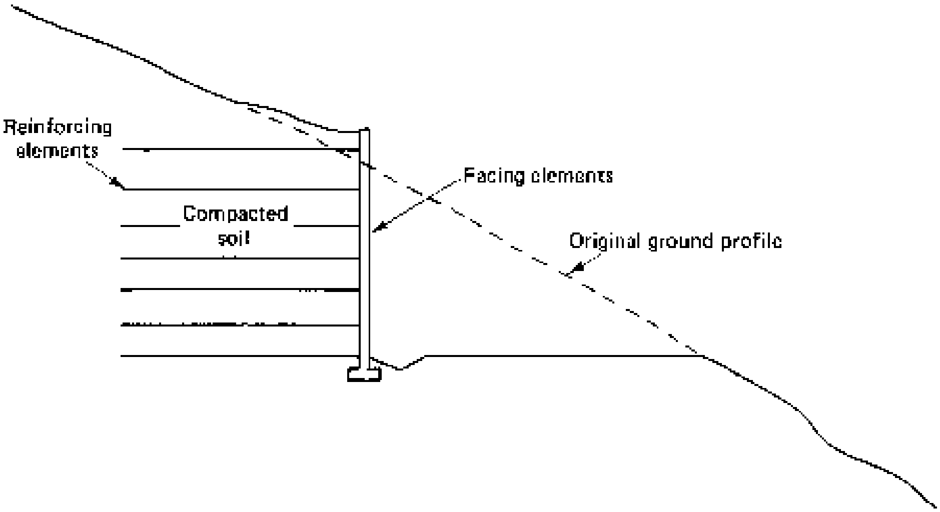


Figure 10–17 Soil reinforcement.

tensile elements are attached to facing material, usually of concrete or timber, to prevent erosion or raveling of soil at the cut surface. Soil reinforcement is often a less expensive method for stabilizing slopes than is the construction of conventional retaining walls.

10–6 PROTECTING EXCAVATIONS AND WORKERS

Excavation cave-ins are responsible for the greatest number of U.S. construction fatalities, accounting for over 300 deaths during a recent year. Because of the frequency and severity of cave-in accidents, OSHA has established a number of safety regulations affecting excavation operations. While it may be possible to avoid placing workers into an excavation through the use of remote-controlled equipment or robots (see Chapter 20), in most cases workers must enter the excavation and OSHA regulations will apply. These regulations require, among other things, that workers in an excavation be protected from cave-ins by one of the following methods:

- Sloping or benching of the sides of the excavation.
- Supporting the sides of the excavation by shoring.
- Placing a shield between workers and the sides of the excavation.

The only exceptions to these requirements are when the excavation is made entirely in stable rock, or the excavation is less than 5 ft (1.524 m) in depth and examination of the ground by a competent person provides no indication of a potential cave-in. As defined by OSHA, *competent person* means one who is capable of identifying existing and predictable hazards in the surroundings, or working conditions which are unsanitary, hazardous, or dangerous to employees, and who has authorization to take prompt corrective measures to eliminate them.

Table 10–1 OSHA soil and rock classification system

Stable Rock	Type A	Type B	Type C
Stable rock means natural solid mineral matter that can be excavated with vertical sides and remain intact while exposed.	Type A means cohesive soil with an unconfined compressive strength of 1.5 tsf (144 kPa) or greater. Examples of cohesive soils are: clay, silty clay, sandy clay, clay loam and, in some cases, silty clay loam and sandy clay loam. Cemented soils such as caliche and hardpan are also considered Type A if: i. The soil is fissured; or ii. The soil is subject to vibration from heavy traffic, pile driving, or similar effects; or iii. The soil has been previously disturbed; or iv. The soil is part of a sloped, layered system where the layers dip into the excavation on a slope of four horizontal to one vertical (4H:1V) or greater; or v. The material is subject to other factors that would require it to be classified as a less stable material.	Type B means: i. Cohesive soil with an unconfined compressive strength greater than 0.5 tsf (48 kPa) but less than 1.5 tsf (144 kPa); or ii. Granular cohesionless soils including angular gravel (similar to crushed rock), silt, silt loam, sandy loam and, in some cases, silty clay loam and sandy clay loam. iii. Previously disturbed soils except those which would otherwise be classed as Type C soil. iv. Soil that meets the unconfined compressive strength or cementation requirements for Type A, but is fissured or subject to vibration; or v. Dry rock that is not stable; or vi. Material that is part of a sloped, layered system where the layers dip into the excavation on a slope less steep than four horizontal to one vertical (4H:1V), but only if the material would otherwise be classified as Type B.	Type C means: i. Cohesive soil with an unconfined compressive strength of 0.5 tsf (48 kPa) or less; or ii. Granular soils including gravel, sand, and loamy sand; or iii. Submerged soil or soil from which water is freely seeping; or iv. Submerged rock that is not stable; or v. Material in a sloped, layered system where the layers dip into the excavation on a slope of four horizontal to one vertical (4H:1V) or steeper.

To comply with OSHA rules on sloping, shoring, and shielding, it is necessary to be familiar with the OSHA Soil and Rock Classification System shown in Table 10–1. In this system, soil and rock are classified as Stable Rock, Type A, Type B, or Type C.

Sloping and Benching

Under OSHA rules, when workers are required to be in an excavation the maximum allowable steepness of the sides of excavations less than 20 ft (6.1 m) deep when employing a simple uniform slope is given in Table 10–2. However, note the exceptions shown in the footnote to the table.

The requirements for benching (stepping) of excavation sides and for sloping when layered soils of different types are involved are given in reference 5. Sloping or benching for excavations greater than 20 ft (6.1 m) deep must be designed by a registered professional engineer. The major disadvantage of sloping or benching of excavation sides is the space required for the excavation plus side slopes.

Table 10–2 OSHA maximum allowable slopes for excavation sides. (From *Code of Federal Regulations*, Part 1926, Title 29, Chapter XVII)

Soil or Rock Type	Maximum Allowable Slope (H:V) for Excavations Less Than 20 ft (6.1 m) Deep
Stable Rock	Vertical (90°)
Type A*	3/4:1 (53°)
Type B	1:1 (45°)
Type C	1-1/2:1 (34°)

*A short-term (24 h or less) maximum allowable slope of 1/2H:1V (63°) is allowed in excavations in Type A soil that are 12 ft (3.67 m) or less in depth. Short-term maximum allowable slopes for excavations greater than 12 ft (3.67 m) in depth shall be 3/4H:1V (53°).

Shoring and Shielding

Lateral support for the sides of an excavation is usually provided by *shoring.* A shoring system that completely encloses an excavation is essentially a cofferdam, which is a structure designed to keep water and/or soil out of an excavation area. A caisson is also a form of cofferdam, as we have seen. Common types of shoring systems include timber shoring, aluminum hydraulic shoring, lagging, and sheet piling. Shoring and shielding systems must be installed in compliance with OSHA tables, manufacturer's tabulated data, or as designed by a registered professional engineer.

Timber shoring (Figure 10–18) employs vertical timber uprights placed against the sides of the excavation, either in a continuous fashion or at intervals. Uprights are supported

Figure 10–18 Timber shoring system.

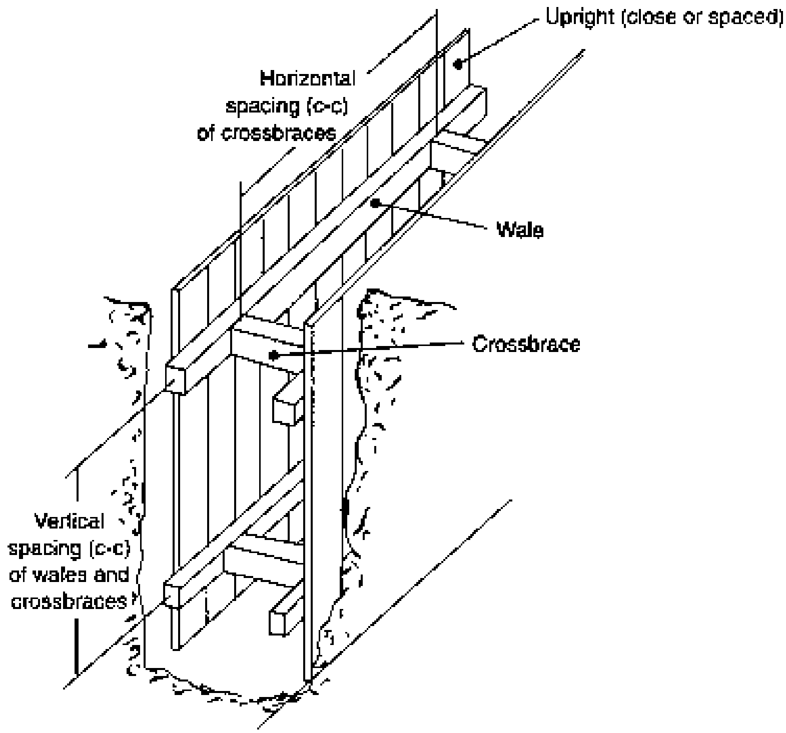
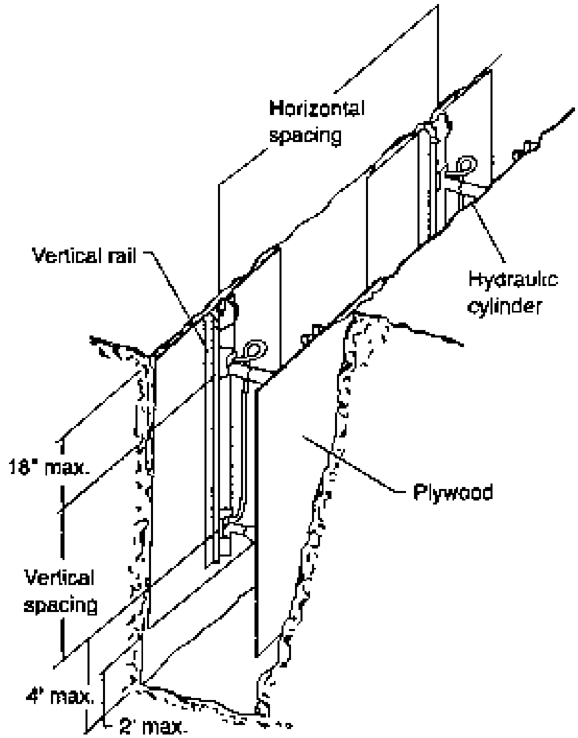


Figure 10–19 Aluminum hydraulic shoring system.



by horizontal beams called wales or stringers. Wales, in turn, are supported by horizontal timber crossbraces or trench jacks. When continuous uprights are used, this shoring system is often called timber sheeting.

Aluminum hydraulic shoring (Figure 10–19) employs prefabricated vertical rails as uprights with attached hydraulic cylinder crossbraces. Plywood sheets may be placed under the vertical uprights as shown to increase the area of support for the excavation sides. Another arrangement uses timber sheeting supported by prefabricated aluminum wales with attached hydraulic cylinder crossbraces. In this case, the shoring system looks very much like the timber shoring system of Figure 10–18.

Lagging is nothing more than sheeting placed horizontally. However, in this case vertical supports (called soldier beams) are required between the lagging and wales. Another lagging system uses soldier piles (such as H-piles) with the lagging placed between the open sides of the piling. Wales and struts or tiebacks are used to provide lateral support.

Sheet piling is sheeting of concrete, steel, or timber that is designed to be driven by a pile driver. Sheet piling is used for constructing retaining walls, shoring, and cofferdams. Two sheet pile walls may be constructed parallel to each other, crossbraced, and filled with earth to form a cofferdam. When tight sheeting or sheet piling is used, the shoring system must be designed to withstand the full hydrostatic pressure of the groundwater level unless weep holes or other drains are provided in the shoring system.

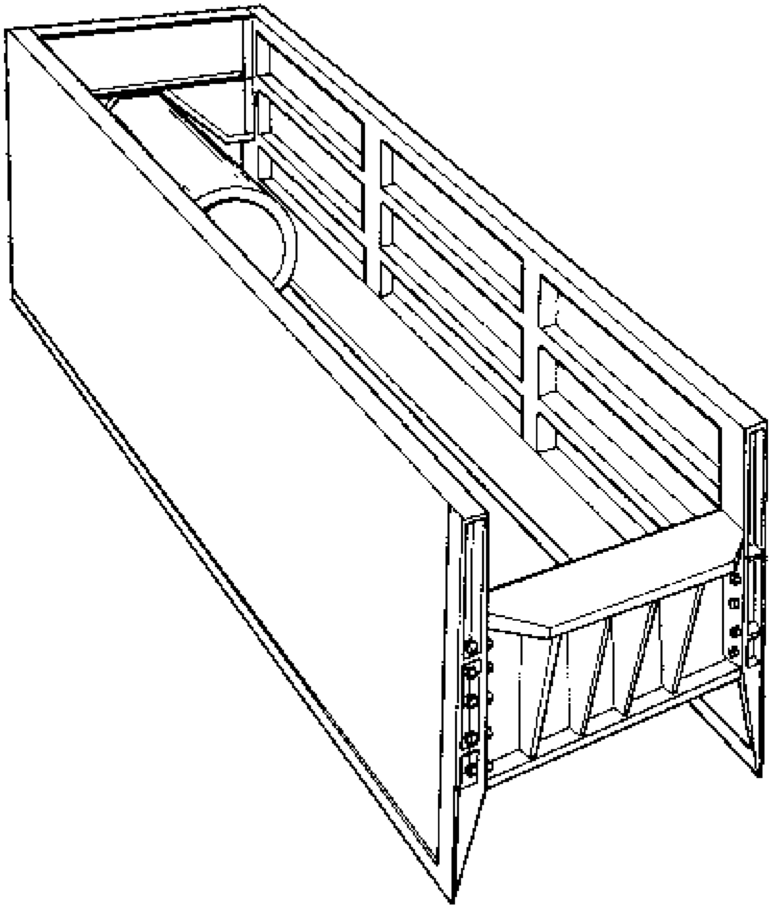


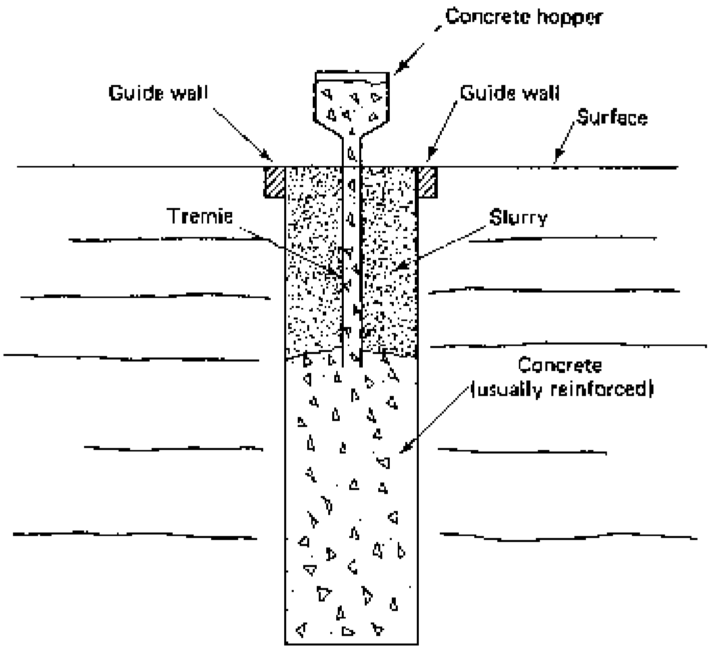
Figure 10–20 Trench shield.

Trench shields or trench boxes are used in place of shoring to protect workers during trenching operations. Figure 10–20 illustrates such a movable trench shield. The top of the shield should extend above the sides of the trench to provide protection for workers against objects falling from the sides of the trench. The trench shield is pulled ahead by the excavator as work progresses.

Slurry Trenches

A relatively new development in excavating and trenching is the construction of *slurry trenches.* In this technique, illustrated in Figure 10–21, a slurry (such as clay and water) is used to fill the excavation as soil is removed. The slurry serves to keep the sides of the trench from collapsing during excavation. No lowering of the water table is required with this method. After completion of the trench, the slurry is displaced by concrete placed

Figure 10–21 Slurry trench construction.



through the slurry by use of a tremie. The slurry is pumped away as it is displaced. The slurry trench technique eliminates the necessity for shoring and dewatering excavations. The soil between two rows of completed slurry trenches may be excavated to form a large opening such as a subway tunnel.

10–7 DEWATERING EXCAVATIONS

Dewatering is the process of removing water from an excavation. Dewatering may be accomplished by lowering the groundwater table before the excavation is begun. This method is often used for placing pipelines in areas with high groundwater levels. Alternatively, excavation may be accomplished first and the water simply pumped out of the excavation as work proceeds. With either procedure, the result is a lowering of the groundwater level in the excavation area. Hence all dewatering methods involve pumping of water from the ground. Keep in mind that lowering of the water table may cause settlement of the soil in the surrounding area. This, in turn, may cause foundation settlement or even foundation failure in buildings near the excavation area.

The selection of an appropriate dewatering method depends on the nature of the excavation and the permeability of the soil. *Soil permeability,* or the ease with which water flows through the soil, is primarily a function of a soil's grain size distribution. It has been found that the diameter of the soil particle which is smaller than 90% of the soil's grains (i.e., 10% of total soil grains are smaller than the designated grain size) is an effective measure of soil permeability. This soil grain size is referred to as the soil's *effective grain*

Table 10–3 Appropriate dewatering methods

Effective Grain Size (D_{10})	Dewatering Method
Larger than 0.1 mm*	Sumps, ordinary wellpoints
0.1–0.004 mm	Vacuum wells or wellpoints
0.004–0.0017 mm	Electroosmosis

*No. 150 sieve size corresponds to an opening of 0.1 mm.

size and is represented by the symbol D_{10}. Table 10–3 indicates appropriate dewatering methods as a function of effective soil grain size. Note that gravity drainage (use of pumps and wellpoints) is effective for soils whose effective grain size is about 0.1 mm (corresponding to a No. 150 sieve size) or larger.

Wellpoint Systems

Figure 10–22 illustrates the use of a standard wellpoint system to dewater an area prior to excavation. Technically, a *wellpoint* is the perforated assembly placed on the bottom of the inlet pipe for a well. It derives its name from the point on its bottom used to facilitate driving the inlet pipe for a well. In practice, the term wellpoint is commonly used to identify each well in a dewatering system, consisting of a number of closely spaced wells. In sandy soils, the usual procedure is to jet the well point and riser into position. This is accomplished by pumping water down through the riser and wellpoint to loosen and liquefy the sand around the wellpoint. Under these conditions, the wellpoint sinks under its own weight to the desired depth. Additional wellpoints are sunk in a line surrounding the excavation area, then connected to a header pipe. Header pipes used for such systems are essentially manifolds consisting of a series of connection points with valves. After all wellpoints are in place and connected to the header, the header pipes are connected to a self-priming centrifugal pump equipped with an air ejector. Since water from the wellpoints is drawn off by creating a partial vacuum at the pump inlet, the maximum height that water can be lifted by the pump is something less than 32 ft (9.8 m). In practice, the maximum effective dewatering depth is about 20 ft (6.1 m) below the ground surface. Wellpoints are typically spaced 2 to 10 ft (0.6–3.1 m) apart and yield flows ranging from 3 to 30 gal/min ($11–114\ \ell/\text{min}$) per wellpoint. Wellpoints placed in very fine sands may require the use of a coarse sand filter around the wellpoint to prevent an excessive flow of fine sand into the system. If the groundwater table must be lowered more than 20 ft (6.1 m) a single stage of wellpoints will not be effective. In this situation, two or more levels of wellpoints (called *stages*) may be used. The major disadvantage of such a system is the large area required for terracing the stages. For example, to lower the water table 36 ft (11 m) using two stages with embankment side slopes of 1 on 2 and allowing a 5-ft-wide (1.5-m-wide) bench for each pump requires a total width of 82 ft (25 m) on each side of the excavation. Alternatives to the use of staged wellpoints include the use of jet pumps and submersible pumps to lift water from the wells. Figure 10–23 shows an electrically powered submersible pump being placed into a dewatering well.

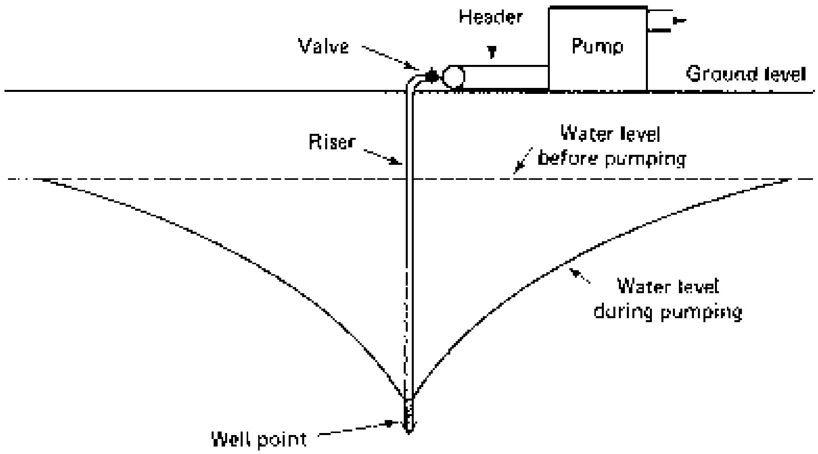


Figure 10–22 Wellpoint dewatering system.

Vacuum Wells

Vacuum wells are wellpoints that are sealed at the surface by placing a ring of bentonite or clay around the well casing. A vacuum pump is then connected to the header pipe. The resulting differential pressure between the well and the surrounding groundwater will accelerate the flow of water into the well. In fine-grained soils, it may also be necessary to place a sand filter around the wellpoint and riser pipe.

Electroosmosis

Electroosmosis is the process of accelerating the flow of water through a soil by the application of a direct current. Although the phenomenon of electroosmosis was discovered in the laboratory early in the nineteenth century, it was not applied to construction dewatering until 1939. As shown in Table 10–3, the method is applicable to relatively impervious soils such as silts and clays having an effective grain size as small as 0.0017 mm.

The usual procedure for employing electroosmosis in dewatering is to space wells at intervals of about 35 ft (10.7 m) and drive grounding rods between each pair of wells. Each well is then connected to the negative terminal of a dc voltage source and each ground rod is connected to a positive terminal. A voltage of 1.5 to 4 V/ft (4.9 to 13 V/m) of distance between the well and anode is then applied, resulting in an increased flow of water to the well (cathode). The applied voltage should not exceed 12 V/ft (39 V/m) of distance between the well and anode to avoid excessive power loss due to heating. Typical current requirements of 15 to 30 A per well result in power demands of 0.5 to 2.5 kW per well.

A measure of the effectiveness of electroosmosis can be gained by comparing the flow developed by an electrical voltage with the flow produced by conventional hydraulic forces. Such a calculation for a clay of average permeability indicates that an electrical potential of 3 V/ft (10 V/m) is equivalent to a hydraulic gradient of 50 ft/ft (50 m/m). To obtain a hydraulic gradient of 50 ft/ft by the use of vacuum wells would require wells to be

Figure 10–23 Electrically powered submersible pump being placed into dewatering well. (Courtesy of Crane Pumps and Systems)

spaced about 1 ft (0.3 m) apart. This calculation provides a measure of the tremendous increase in water flow that electroosmosis provides in soils of low permeability over the flow produced by conventional hydraulic methods.

10–8 PRESSURE GROUTING

Grouting or *pressure grouting* is the process of injecting a grouting agent into soil or rock to increase its strength or stability, protect foundations, or reduce groundwater flow. Grouting of rock is widely employed in dam construction and tunneling. The need for such grouting is determined by exploratory methods such as core drilling and visual observation in test holes. Pressure tests that measure the flow of water through injector pipes which have been placed and sealed into test holes may also be employed as a measure of the need for grouting and for measuring the effectiveness of grouting. Recent developments in grouting agents and injection methods have led to an increasing use of grouting in soils.

Common grouting patterns include blanket grouting, curtain grouting, and special grouting. *Blanket grouting* covers a large horizontal area, usually to a depth of 50 ft (15 m) or less. *Curtain grouting* produces a linear deep, narrow zone of grout that may extend to a depth of 100 ft (30 m) or more. It is commonly employed to form a deep barrier to water flow under a dam. *Special grouting* is grouting employed for a specific purpose, such as to consolidate rock or soil around a tunnel, fill individual rock cavities, or provide additional foundation support.

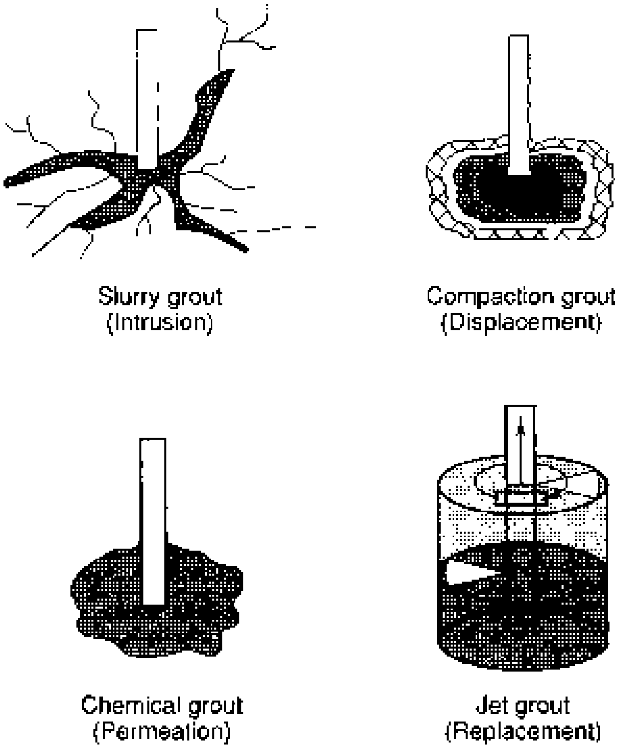


Figure 10–24 Types of grouting. (Courtesy of Hayward Baker Inc., A Keller Company)

Grouting Methods

Major types of grouting include slurry grouting, chemical grouting, compaction grouting, and jet grouting (Figure 10–24).

Slurry grouting involves the injection of a slurry consisting of water and a grouting agent into soil or rock. Common grouting materials include portland cement, clay (bentonite), fly ash, sand, lime, and additives. In soil, regular portland cement grouts are able to effectively penetrate only gravel and coarse sand. Newer *microfine cement* (or fine-grind cement) grouts are able to penetrate medium and fine sands. Injection of *lime slurry* grout can be used to control the swelling of expansive clays. It can also be used to stabilize low-strength soils such as silts, dredge spoil, and saturated soils.

Chemical grouting involves the injection of a chemical into soil. It is used primarily in sands and fine gravel to cement the soil particles together for structural support or to control water flow. The proper selection of a chemical grout and additives permit rather precise control of grout hardening (setting) time.

Compaction grouting is the process of injecting a very stiff mortar grout into a soil to compact and strengthen the soil. Grouting materials include silty sand, cement, fly ash, additives, and water. Compaction grouting is able to create grout bulbs or grout piles in the soil which serve to densify the soil and provide foundation support. Compaction

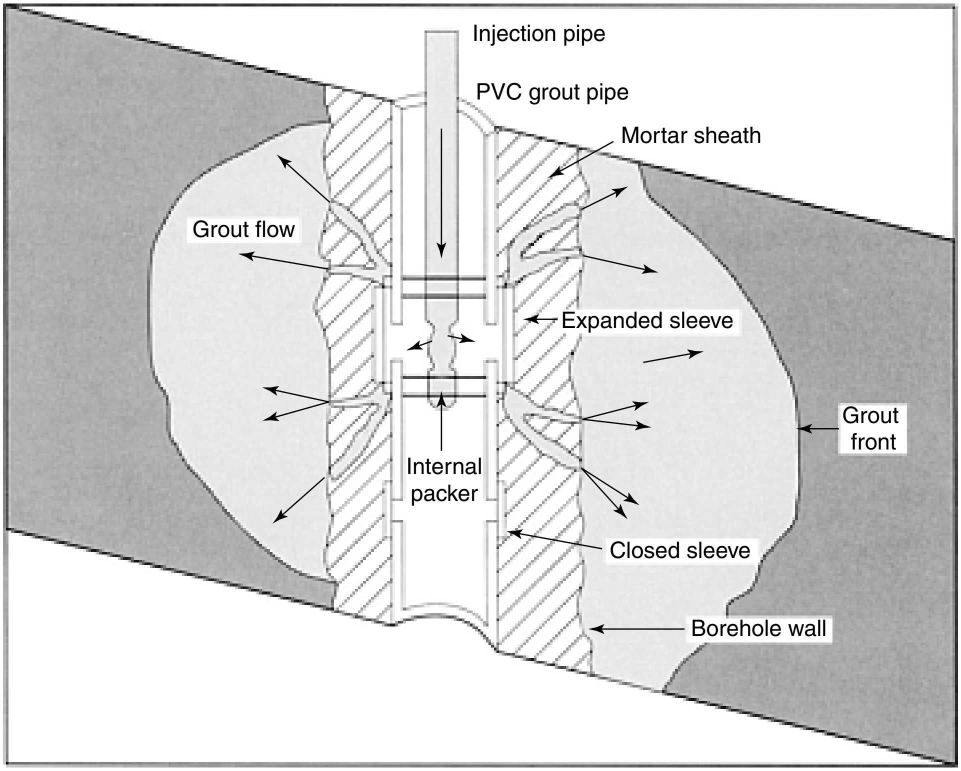


Figure 10–25 Grouting utilizing a sleeve port pipe. (Courtesy of Hayward Baker, Inc., A Keller Company)

grouting can also be used to raise (jack) foundations that have settled back to their original elevation.

Jet grouting employs a rotating jet pipe to remove soil around the grout pipe and replace the soil with grout. As a result, the technique is effective over a wide range of soil types to include silts and some clays. The unconfined compressive strength of the grouted soil structure may run as high as 2500 lb/in.^{2} (17 MPa).

Injection Methods

The principal method for injecting grout into rock involves drilling a hole and then inserting an injector pipe equipped with expandable seals (*packers*) into the hole. Grout is then injected at the desired depths. Methods for injecting grout into soil include driving an injector pipe into the soil, placing a sleeve port tube into the soil, and jet grouting. Grouting utilizing a sleeve port pipe is illustrated in Figure 10–25. Notice that the grout pipe is equipped with sleeves that cover ports spaced at intervals along the pipe. The sleeves serve as check valves to allow grout to flow out of the ports but prevent return flow. Packers serve to direct the flow of grout through the desired ports.

Selection of an optimum grouting agent and grouting system should be accomplished by experienced grouting specialists. Trial grouting and testing will usually be required before selecting the grouting system to be employed. Care must be taken to avoid the use of injection pressures that lift the ground surface, unless a jacking action is desired.

PROBLEMS

1. Briefly describe the process of installing a shell pile.
2. Briefly describe and contrast typical slope failure in a pure cohesionless soil and a pure cohesive soil.
3. Using Equation 10–2 and the following driving data, determine the safe load of an 8-in.-square concrete pile 40 ft long. Assume that the unit weight of the pile is 150 lb/cu ft.

   Pile Driver:

   Single-acting compressed air hammer

   Rated energy = 15,000 ft-lb

   Ram weight = 5000 lb

   Weight of driving appurtenances = 2000 lb

   Average penetration last six blows = 1/4 in./blow

4. Briefly describe the four major types of pressure grouting.
5. Briefly explain the process of vibratory compaction as used in soil improvement.
6. Explain the significance of a soil's effective grain size (D_{10}) to the process of dewatering the soil.
7. How does a pile support its applied load?
8. When sloping the sides of an excavation in type A soil, what maximum slope may be used if the excavation will be 15 ft (4.6 m) deep and will be open less than 24 h?
9. What three methods meet OSHA requirements for protecting workers in an excavation when worker protection against cave-in is required?
10. Write a computer program to predict the safe capacity of a pile driven by a powered hammer using Equation 10–2. Solve Problem 3 using your computer program.

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