SECTION SIX
SOIL MECHANICS AND FOUNDATIONS

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6.1 INTRODUCTION

6.1.1 Soil Mechanics

Soil mechanics is defined as the application of the laws and principles of mechanics and hydraulics to engineering problems dealing with soil as an engineering material. Soil has many different meanings, depending on the field of study. For example, in agronomy (application of science to farming), soil is defined as a surface deposit that contains mineral matter that originated from the original weathering of rock and also contains organic matter that has accumulated through the decomposition of plants and animals. To an agronomist, soil is that material that has been sufficiently altered and supplied with nutrients that it can support the growth of plant roots. But to a geotechnical engineer, soil has a much broader meaning and can include not only agronomic material, but also broken-up fragments of rock, volcanic ash, alluvium, aeolian sand, glacial material, and any other residual or transported product of rock weathering. Difficulties naturally arise because there is not a distinct dividing line between rock and soil. For example, to a geologist a given material may be classified as a formational rock because it belongs to a definite geologic environment, but to a geotechnical engineer it may be sufficiently weathered or friable that it should be classified as a soil.

6.1.2 Rock Mechanics

Rock mechanics is defined as the application of the knowledge of the mechanical behavior of rock to engineering problems dealing with rock. To the geotechnical engineer, rock is a relatively solid mass that has permanent and strong bonds between the minerals. Rocks can be classified as being either sedimentary, igneous, or metamorphic. There are significant differences in the behavior of soil versus rock, and there is not much overlap between soil mechanics and rock mechanics.

Table 6.1 presents a list of common soil and rock conditions that require special consideration by the geotechnical engineer.

6.1.3 Foundation Engineering

A **foundation** is defined as that part of the structure that supports the weight of the structure and transmits the load to underlying soil or rock. Foundation engineering applies the knowledge of soil mechanics, rock mechanics, geology, and

TABLE 6.1 Problem Conditions Requiring Special Consideration

Problem type	Description	Comments
Soil	Organic soil, highly plastic soil	Low strength and high compressibility
	Sensitive clay	Potentially large strength loss upon large straining
	Micaceous soil	Potentially high compressibility
	Expansive clay, silt, or slag	Potentially large expansion upon wetting
	Liquefiable soil	Complete strength loss and high deformations caused by earthquakes
	Collapsible soil	Potentially large deformations upon wetting
	Pyritic soil	Potentially large expansion upon oxidation
Rock	Laminated rock	Low strength when loaded parallel to bedding
	Expansive shale	Potentially large expansion upon wetting; degrades readily upon exposure to air and water
	Pyritic shale	Expands upon exposure to air and water
	Soluble rock	Rock such as limestone, limerock, and gypsum that is soluble in flowing and standing water
	Cretaceous shale	Indicator of potentially corrosive groundwater
	Weak claystone	Low strength and readily degradable upon exposure to air and water
	Gneiss and schist	Highly distorted with irregular weathering profiles and steep discontinuities
	Subsidence	Typical in areas of underground mining or high groundwater extraction
	Sinkholes	Areas underlain by carbonate rock (Karst topography)
Condition	Negative skin friction	Additional compressive load on deep foundations due to settlement of soil
	Expansion loading	Additional uplift load on foundation due to swelling of soil
	Corrosive environment	Acid mine drainage and degradation of soil and rock
	Frost and permafrost	Typical in northern climates
	Capillary water	Rise in water level which leads to strength loss for silts and fine sands

Source: ''Standard Specifications for Highway Bridges,'' 16th ed., American Association of State Highway and Transporation Officials, Washington, DC.

structural engineering to the design and construction of foundations for buildings and other structures. The most basic aspect of foundation engineering deals with the selection of the type of foundation, such as using a shallow or deep foundation system. Another important aspect of foundation engineering involves the development of design parameters, such as the bearing capacity of the foundation. Foundation engineering could also include the actual foundation design, such as determining the type and spacing of steel reinforcement in concrete footings. As indicated in Table 6.2, foundations are commonly divided into two categories: shallow and deep foundations.

6.2 FIELD EXPLORATION

The purpose of the field exploration is to obtain the following (M. J. Tomlinson, ''Foundation Design and Construction,'' 5th ed., John Wiley & Sons, Inc., New York):

1. Knowledge of the general topography of the site as it affects foundation design and construction, e.g., surface configuration, adjacent property, the presence of watercourses, ponds, hedges, trees, rock outcrops, etc., and the available access for construction vehicles and materials.
2. The location of buried utilities such as electric power and telephone cables, water mains, and sewers.
3. The general geology of the area, with particular reference to the main geologic formations underlying the site and the possibility of subsidence from mineral extraction or other causes.
4. The previous history and use of the site, including information on any defects or failures of existing or former buildings attributable to foundation conditions.
5. Any special features such as the possibility of earthquakes or climate factors such as flooding, seasonal swelling and shrinkage, permafrost, and soil erosion.
6. The availability and quality of local construction materials such as concrete aggregates, building and road stone, and water for construction purposes.
7. For maritime or river structures, information on tidal ranges and river levels, velocity of tidal and river currents, and other hydrographic and meteorological data.
8. A detailed record of the soil and rock strata and groundwater conditions within the zones affected by foundation bearing pressures and construction operations, or of any deeper strata affecting the site conditions in any way.
9. Results of laboratory tests on soil and rock samples appropriate to the particular foundation design or construction problems.
10. Results of chemical analyses on soil or groundwater to determine possible deleterious effects of foundation structures.

6.2.1 Document Review

Some of the required information, such as the previous history and use of the site, can be obtained from a document review. For example, there may be old engi-

TABLE 6.2 Common Types of Foundations

Category	Common types	Comments
Shallow foundations	Spread footings (also called pad footings)	Spread footings are often square in plan view, are of uniform reinforced concrete thickness, and are used to support a single column load located directly in the center of the footing.
	Strip footings (also called wall footings)	Strip or wall footings are often used for load-bearing walls. They are usually long reinforced concrete members of uniform width and shallow depth.
	Combined footings	Reinforced concrete combined footings that carry more than one column load are often rectangular or trapezoidal in plan view.
	Conventional slab-on-grade	A continuous reinforced concrete foundation consisting of bearing wall footings and a slab-on-grade. Concrete reinforcement often consists of steel re-bar in the footings and wire mesh in the concrete slab.
	Post-tensioned slab-on-grade	A continuous post-tensioned concrete foundation. The post-tensioning effect is created by tensioning steel tendons or cables embedded within the concrete. Common post-tensioned foundations are the ribbed foundation, California Slab, and PTI foundation.
	Raised wood floor	Perimeter footings that support wood beams and a floor system. Interior support is provided by pad or strip footings. There is a crawl space below the wood floor.
	Mat foundation	A large and thick reinforced concrete foundation, often of uniform thickness, that is continuous and supports the entire structure. A mat foundation is considered to be a shallow foundation if it is constructed at or near ground surface.

TABLE 6.2 Common Types of Foundations (Continued)

Category	Common types	Comments
Deep foundations	Driven piles	Driven piles are slender members, made of wood, steel, or precast concrete, that are driven into place using pile-driving equipment.
	Other types of piles	There are many other types of piles, such as bored piles, cast-in-place piles, or composite piles.
	Piers	Similar to cast-in-place piles, piers are often of large diameter and contain reinforced concrete. Pier and grade beam support are often used for foundation support on expansive soil.
	Caissons	Large piers are sometimes referred to as caissons. A caisson can also be a watertight underground structure within which construction work is carried on.
	Mat or raft foundation	If a mat or raft foundation is constructed below ground surface or if the mat or raft foundation is supported by piles or piers, then it should be considered to be a deep foundation system.
	Floating foundation	A special foundation type where the weight of the structure is balanced by the removal of soil and construction of an underground basement.
	Basement-type foundation	A common foundation for houses and other buildings in frost-prone areas. The foundation consists of perimeter footings and basement walls that support a wood floor system. The basement floor is usually a concrete slab.

Shallow and deep foundations in this table are based on the depth of the soil or rock support of the foundation.

neering reports indicating that the site contains deposits of fill, abandoned septic systems and leach fields, buried storage tanks, seepage pits, cisterns, mining shafts, tunnels, or other man-made surface and subsurface works that could impact the new proposed development. There may also be information concerning on-site utilities and underground pipelines, which may need to be capped or rerouted around the project.

During the course of the work, it may be necessary to check reference materials, such as geologic and topographic maps. Geologic maps can be especially useful because they often indicate potential geologic hazards (e.g., faults, landslides) as well as the type of near-surface soil or rock at the site. Both old and recent topographic maps can also provide valuable site information. Topographic maps are usually to scale and show the locations of buildings, roads, freeways, train tracks, and other civil engineering works as well as natural features such as canyons, rivers, lagoons, sea cliffs, and beaches. The topographic maps can even show the locations of sewage disposal ponds and water tanks, and by using different colors and shading, they indicate older versus newer development. But the main purpose of the topographic map is to indicate ground surface elevations. This information can be used to determine the major topographic features at the site and for the planning of subsurface exploration, such as available site access for drilling rigs.

Another important source of information is aerial photographs, which are taken from an aircraft flying at a prescribed altitude along preestablished lines. Viewing a pair of aerial photographs, with the aid of a stereoscope, provides a three-dimensional view of the land surface. This view may reveal important geologic information at the site, such as the presence of landslides, fault scarps, types of landforms (e.g., dunes, alluvial fans, glacial deposits such as moraines and eskers), erosional features, general type and approximate thickness of vegetation, and drainage patterns. By comparing older versus newer aerial photographs, the engineering geologist can also observe any man-made or natural changes that have occurred at the site.

6.2.2 Subsurface Exploration

In order for a detailed record of the soil and rock strata and groundwater conditions at the site to be determined, subsurface exploration is usually required. There are different types of subsurface exploration, such as borings, test pits, and trenches. Table 6.3 summarizes the boring, core drilling, sampling, and other exploratory techniques that can be used by the geotechnical engineer.

A **boring** is defined as a cylindrical hole drilled into the ground for the purposes of investigating subsurface conditions, performing field tests, and obtaining soil, rock, or groundwater specimens for testing. Borings can be excavated by hand (e.g., with a hand auger), although the usual procedure is to use mechanical equipment to excavate the borings.

Many different types of equipment are used to excavate borings. Typical types of borings are listed in Table 6.3 and include:

Auger Boring. A mechanical auger is a very fast method of excavating a boring. The hole is excavated by rotating the auger while at the same time applying a downward pressure on the auger to help obtain penetration of the soil or rock. There are basically two types of augers: **flight augers** and **bucket augers.** Common available diameters of flight augers are 5 cm to 1.2 m (2 in to 4 ft) and of bucket augers are 0.3 m to 2.4 m (1 ft to 8 ft). The auger is periodically removed

TABLE 6.3 Boring, Core Drilling, Sampling, and Other Exploratory Techniques*

Method (1)	Procedure (2)	Type of sample (3)	Applications (4)	Limitations (5)
Auger boring, ASTM D 1452	Dry hole drilled with hand or power auger; samples preferably recovered from auger flutes	Auger cuttings, disturbed, ground up, partially dried from drill heat in hard materials	In soil and soft rock; to identify geologic units and water content above water table	Soil and rock stratification destroyed; sample mixed with water below the water table
Test boring, ASTM D 1586	Hole drilled with auger or rotary drill; at intervals samples taken 36-mm (1.4-in) ID and 50-mm (2-in) OD driven 0.45 m (1.5 ft) in three 150-mm (6-in) increments by 64-kg (140-lb) hammer falling 0.76 m (30 in); hydrostatic balance of fluid maintained below water level	Intact but partially disturbed (number of hammer blows for second plus third increment of driving is standard penetration resistance or *N*)	To identify soil or soft rock; to determine water content; in classification tests and crude shear test of sample (*N*-value a crude index to density of cohesionless soil and undrained shear strength of cohesive soil)	Gaps between samples, 30 to 120 cm (12 to 50 in); sample too distorted for accurate shear and consolidation tests; sample limited by gravel; *N*-value subject to variations, depending on free fall of hammer
Test boring of large samples	50- to 75-mm (2- to 3-in) ID and 63- to 89-mm (2.5- to 3.5-in) OD samplers driven by hammers up to 160 kg (350 lb)	Intact but partially disturbed (number of hammer blows for second plus third increment of driving is penetration resistance)	In gravelly soils	Sample limited by larger gravel
Test boring through hollow stem auger	Hole advanced by hollow stem auger; soil sampled below auger as in test boring above	Intact but partially disturbed (number of hammer blows for second plus third increment of driving is *N*-value)	In gravelly soils (not well adapted to harder soils or soft rock)	Sample limited by larger gravel; maintaining hydrostatic balance in hole below water table is difficult

TABLE 6.3 Boring, Core Drilling, Sampling, and Other Exploratory Techniques* (*Continued*)

Method (1)	Procedure (2)	Type of sample (3)	Applications (4)	Limitations (5)
Rotary coring of soil or soft rock	Outer tube with teeth rotated; soil protected and held stationary in inner tube; cuttings flushed upward by drill fluid (examples: Denison, Pitcher, and Acker samplers)	Relatively undisturbed sample, 50 to 200 mm (2 to 8 in) wide and 0.3 to 1.5 m (1 to 5 ft) long in liner tube	In firm to stiff cohesive soils and soft but coherent rock	Sample may twist in soft clays; sampling loose sand below water table is difficult; success in gravel seldom occurs
Rotary coring of swelling clay, soft rock	Similar to rotary coring of rock; swelling core retained by third inner plastic liner	Soil cylinder 28.5 to 53.2 mm (1.1 to 2.0 in) wide and 600 to 1500 mm (24 to 60 in) long, encased in plastic tube	In soils and soft rocks that swell or disintegrate rapidly in air (protected by plastic tube)	Sample smaller; equipment more complex
Rotary coring of rock, ASTM D 2113	Outer tube with diamond bit on lower end rotated to cut annular hole in rock; core protected by stationary inner tube; cuttings flushed upward by drill fluid	Rock cylinder 22 to 100 mm (0.9 to 4 in) wide and as long as 6 m (20 ft), depending on rock soundness	To obtain continuous core in sound rock (percent of core recovered depends on fractures, rock variability, equipment, and driller skill)	Core lost in fractured or variable rock; blockage prevents drilling in badly fractured rock; dip of bedding and joint evident but not strike
Rotary coring of rock, oriented core	Similar to rotary coring of rock above; continuous grooves scribed on rock core with compass direction	Rock cylinder, typically 54 mm (2 in) wide and 1.5 m (5 ft) long with compass orientation	To determine strike of joints and bedding	Method may not be effective in fractured rock

TABLE 6.3 Boring, Core Drilling, Sampling, and Other Exploratory Techniques* (*Continued*)

Method (1)	Procedure (2)	Type of sample (3)	Applications (4)	Limitations (5)
Rotary coring of rock, wire line	Outer tube with diamond bit on lower end rotated to cut annular hole in rock; core protected by stationary inner tube; cuttings flushed upward by drill fluid; core and stationary inner tube retrieved from outer core barrel by lifting device or ‘‘overshot’’ suspended on thin cable (wire line) through special large-diameter drill rods and outer core barrel	Rock cylinder 36.5 to 85 mm (1.4 to 3.3 in) wide and 1.5 to 4.6 m (5 to 15 ft) long	To recover core better in fractured rock, which has less tendency for caving during core removal; to obtain much faster cycle of core recovery and resumption of drilling in deep holes	Same as ASTM D 2113 but to lesser degree
Rotary coring of rock, integral sampling method	22-mm (0.9-in) hole drilled for length of proposed core; steel rod grouted into hole; core drilled around grouted rod with 100- to 150-mm (4- to 6-in) rock coring drill (same as for ASTM D 2113)	Continuous core reinforced by grouted steel rod	To obtain continuous core in badly fractured, soft, or weathered rock in which recovery is low by ASTM D 2113	Grout may not adhere in some badly weathered rock; fractures sometimes cause drift of diamond bit and cutting rod
Thin-wall tube, ASTM D 1587	75- to 1250-mm (3–50 in) thin-wall tube forced into soil with static force (or driven in soft rock); retention of sample helped by drilling mud	Relatively undisturbed sample, length 10 to 20 diameters	In soft to firm clays, short (5-diameter) samples of stiff cohesive soil, soft rock and, with aid of drilling mud, in firm to dense sands	Cutting edge wrinkled by gravel; samples lost in loose sand or very soft clay below water table; more disturbance occurs if driven with hammer

TABLE 6.3 Boring, Core Drilling, Sampling, and Other Exploratory Techniques* (*Continued*)

Method (1)	Procedure (2)	Type of sample (3)	Applications (4)	Limitations (5)
Thin-wall tube, fixed piston	75- to 1250-mm (3- to 50-in) thin-wall tube, which has internal piston controlled by rod and keeps loose cuttings from tube, remains stationary while outer thin-wall tube forced ahead into soil; sample in tube is held in tube by aid of piston	Relatively undisturbed sample, length 10 to 20 diameters	To minimize disturbance of very soft clays (drilling mud aids in holding samples in loose sand below water table)	Method is slow and cumbersome
Swedish foil	Samples surrounded by thin strips of stainless steel, stored above cutter, to prevent contact of soil with tube as it is forced into soil	Continuous samples 50 mm (2 in) wide and as long as 12 m (40 ft)	In soft, sensitive clays	Samples sometimes damaged by coarse sand and fine gravel
Dynamic sounding	Enlarged disposable point on end of rod driven by weight falling fixed distance in increments of 100 to 300 mm (4 to 12 in)	None	To identify significant differences in soil strength or density	Misleading in gravel or loose saturated fine cohesionless soils
Static penetration	Enlarged cone, 36 mm (1.4 in) diameter and 60° angle forced into soil; force measured at regular intervals	None	To identify significant differences in soil strength or density; to identify soil by resistance of friction sleeve	Stopped by gravel or hard seams

TABLE 6.3 Boring, Core Drilling, Sampling, and Other Exploratory Techniques* (*Continued*)

Method (1)	Procedure (2)	Type of sample (3)	Applications (4)	Limitations (5)
Borehole camera	Inside of core hole viewed by circular photograph or scan	Visual representation	To examine stratification, fractures, and cavities in hole walls	Best above water table or when hole can be stabilized by clear water
Pits and trenches	Pit or trench excavated to expose soils and rocks	Chunks cut from walls of trench; size not limited	To determine structure of complex formations; to obtain samples of thin critical seams such as failure surface	Moving excavation equipment to site, stabilizing excavation walls, and controlling groundwater may be difficult
Rotary or cable tool well drill	Toothed cutter rotated or chisel bit pounded and churned	Ground	To penetrate boulders, coarse gravel; to identify hardness from drilling rates	Identifying soils or rocks difficult
Percussion drilling (jack hammer or air track)	Impact drill used; cuttings removed by compressed air	Rock dust	To locate rock, soft seams, or cavities in sound rock	Drill becomes plugged by wet soil

Source: G. F. Sowers and D. L. Royster, ''Field Investigation,'' ch. 4 of ''Landslides: Analysis and Control, Special Report 176,'' ed. R. L. Schuster and R. J. Krizek, National Academy of Sciences, Washington, DC.

from the hole, and the soil lodged in the groves of the flight auger or contained in the bucket of the bucket auger is removed. A casing is generally not used for auger borings, and the hole may cave-in during the excavation of loose or soft soils or when the excavation is below the groundwater table. Augers are probably the most common type of equipment used to excavate borings.

Hollow-Stem Flight Auger. A hollow-stem flight auger has a circular hollow core which allows for sampling down the center of the auger. The hollow-stem auger acts like a casing and allows for sampling in loose or soft soils or when the excavation is below the groundwater table.

Wash-Type Borings. Wash-type borings use circulating drilling fluid, which removes cuttings from the borehole. The cuttings are created by the chopping, twisting, and jetting action of the drill bit, which breaks the soil or rock into small fragments. Casings are often used to prevent cave-in of the hole. Because drilling fluid is used during the excavation, it can be difficult to classify the soil and obtain uncontaminated soil samples.

Rotary Coring. This type of boring equipment uses power rotation of the drilling bit as circulating fluid removes cuttings from the hole. Table 6.3 lists various types of rotary coring for soil and rock.

Percussion Drilling. This type of drilling equipment is often used to penetrate hard rock, for subsurface exploration or for the purpose of drilling wells. The drill bit works much like a jackhammer, rising and falling to break up and crush the rock material.

In addition to borings, other methods for performing subsurface exploration include test pits and trenches. Test pits are often square in plan view, with a typical dimension of 1.2 m by 1.2 m (4 ft by 4 ft). Trenches are long and narrow excavations usually made by a backhoe or bulldozer. Table 6.4 presents the uses, capabilities, and limitations of test pits and trenches.

Test pits and trenches provide for a visual observation of subsurface conditions. They can also be used to obtain undisturbed block samples of soil. The process consists of carving a block of soil from the side or bottom of the test pit or trench. Soil samples can also be obtained from the test pits or trenches by manually driving Shelby tubes, drive cylinders, or other types of sampling tubes into the ground. (See Art. 6.2.3.)

Backhoe trenches are an economical means of performing subsurface exploration. The backhoe can quickly excavate the trench, which can then be used to observe and test the in-situ soil. In many subsurface explorations, backhoe trenches are used to evaluate near-surface and geologic conditions (i.e., up to 15 ft deep), with borings being used to investigate deeper subsurface conditions.

6.2.3 Soil Sampling

Many different types of samplers are used to retrieve soil and rock specimens from the borings. Common examples are indicated in Table 6.3. Figure 6.1 shows three types of samplers, the ‘‘California Sampler,’’ Shelby tube sampler, and Standard Penetration Test (SPT) sampler.

The most common type of soil sampler used in the United States is the Shelby tube, which is a thin-walled sampling tube. It can be manufactured to different diameters and lengths, with a typical diameter varying from 5 to 7.6 cm (2 to 3 in) and a length of 0.6 to 0.9 m (2 to 3 ft). The Shelby tube should be manufactured

TABLE 6.4 Use, Capabilities, and Limitations of Test Pits and Trenches

Exploration method	General use	Capabilities	Limitations
Hand-excavated test pits	Bulk sampling, in-situ testing, visual inspection	Provides data in inaccessible areas, less mechanical disturbance of surrounding ground	Expensive, time-consuming, limited to depths above groundwater level
Backhoe-excavated test pits and trenches	Bulk sampling, in-situ testing, visual inspection, excavation rates, depth of bedrock and groundwater	Fast, economical, generally less than 4.6 m (15 ft) deep, can be up to 9 m (30 ft) deep	Equipment access, generally limited to depths above groundwater level, limited undisturbed sampling
Dozer cuts	Bedrock characteristics, depth of bedrock and groundwater level, rippability, increase depth capability of backhoe, level area for other exploration equipment	Relatively low cost, exposures for geologic mapping	Exploration limited to depth above the groundwater table
Trenches for fault investigations	Evaluation of presence and activity of faulting and sometimes landslide features	Definitive location of faulting, subsurface observation up to 9 m (30 ft) deep	Costly, time-consuming, requires shoring, only useful where dateable materials are present, depth limited to zone above the groundwater level

Source: NAVFAC DM-7.1, 1982.

to meet exact specifications, such as those stated by ASTM D 1587-94 (1998). The Shelby tube shown in Fig. 6.1 has an inside diameter of 6.35 cm (2.5 in).

Many localities have developed samplers that have proven successful with local soil conditions. For example, in southern California, a common type of sampler is the California Sampler, which is a split-spoon type sampler that contains removable internal rings, 2.54 cm (1 in) in height. Figure 6.1 shows the California Sampler in an open condition, with the individual rings exposed. The California Sampler has a 7.6-cm (3.0 in) outside diameter and a 6.35-cm (2.50-in) inside diameter. This sturdy sampler, which is considered to be a thick-walled sampler, has proven successful in sampling hard and desiccated soil and soft sedimentary rock common in southern California.

Three types of soil samples can be recovered from borings:

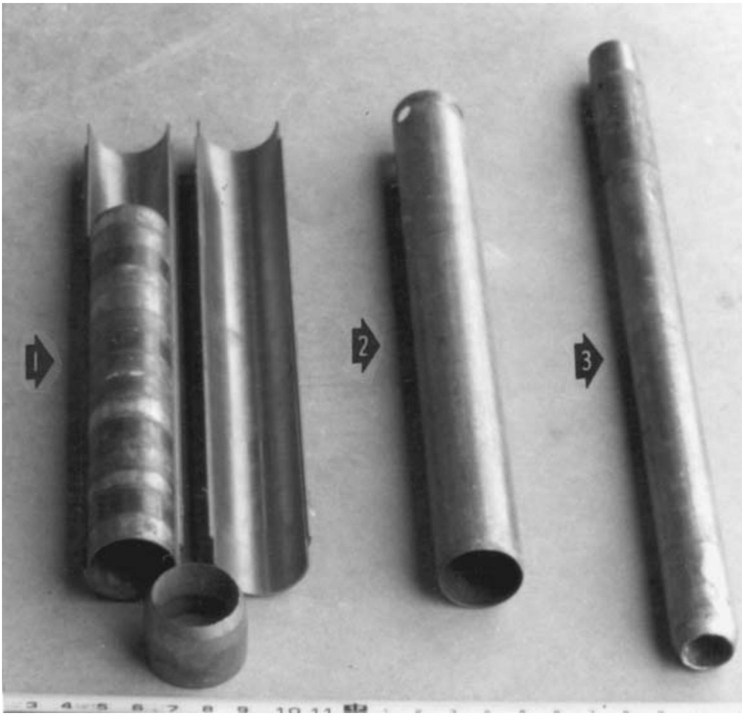


FIGURE 6.1 Soil Samplers (no. 1 is the California Sampler in an open condition, no. 2 is a Shelby Tube, and no. 3 is the Standard Penetration Test sampler.)

1. Altered Soil. During the boring operations, soil can be altered due to mixing or contamination. For example, if the boring is not cleaned out prior to sampling, a soil sample taken from the bottom of the borehole may actually consist of cuttings from the side of the borehole. These borehole cuttings, which have fallen to the bottom of the borehole, will not represent in-situ conditions at the depth sampled. In other cases, the soil sample may become contaminated with drilling fluid, which is used for wash-type borings. These types of soil samples that have been mixed or contaminated by the drilling process should not be used for laboratory tests because they will lead to incorrect conclusions regarding subsurface conditions. Soil that has a change in moisture content due to the drilling fluid or heat generated during the drilling operations should also be classified as altered soil. Soil that has been densified by over-pushing or over-driving the soil sampler should also be considered as altered because the process of over-pushing or over-driving could squeeze water from the soil.

2. Disturbed Samples. Disturbed soil is defined as soil that has been remolded during the sampling process. For example, soil obtained from driven samplers, such as the Standard Penetration Test spilt spoon sampler, or chunks of intact soil brought to the surface in an auger bucket (i.e., bulk samples), are considered disturbed soil. Disturbed soil can be used for numerous types of laboratory tests.

3. Undisturbed Sample. It should be recognized that no soil sample can be taken from the ground in a perfectly undisturbed state. However, this terminology has been applied to those soil samples taken by certain sampling methods. Undisturbed samples are often defined as those samples obtained by slowly pushing thin-walled tubes, having sharp cutting ends and tip relief, into the soil. Two parameters, the **inside clearance ratio** and the **area ratio,** are often used to evaluate the disturbance potential of different samplers, and they are defined as follows:

$$\text{inside clearance ratio (\%)} = 100\,\frac{D_i - D_e}{D_e} \tag{6.1}$$

$$\text{area ratio (\%)} = 100\,\frac{D_o^2 - D_i^2}{D_i^2} \tag{6.2}$$

where D_e = diameter at the sampler cutting tip
D_i = inside diameter of the sampling tube
D_o = outside diameter of the sampling tube

In general, a sampling tube for undisturbed soil specimens should have an inside clearance ratio of about 1% and an area ratio of about 10% or less. Having an inside clearance ratio of about 1% provides for tip relief of the soil and reduces the friction between the soil and inside of the sampling tube during the sampling process. A thin film of oil can be applied at the cutting edge to also reduce the friction between the soil and metal tube during sampling operations. The purpose of having a low area ratio and a sharp cutting end is to slice into the soil with as little disruption and displacement of the soil as possible. Shelby tubes are manufactured to meet these specifications and are considered to be undisturbed soil samplers. As a comparison, the California Sampler has an area ratio of 44% and is considered to be a thick-walled sampler.

It should be mentioned that using a thin-walled tube, such as a Shelby tube, will not guarantee an undisturbed soil specimen. Many other factors can cause soil disturbance, such as:

- Pieces of hard gravel or shell fragments in the soil, which can cause voids to develop along the sides of the sampling tube during the sampling process
- Soil adjustment caused by stress relief when making a borehole
- Disruption of the soil structure due to hammering or pushing the sampling tube into the soil stratum
- Expansion of gas during retrieval of the sampling tube
- Jarring or banging the sampling tube during transportation to the laboratory
- Roughly removing the soil from the sampling tube
- Crudely cutting the soil specimen to a specific size for a laboratory test

The actions listed above cause a decrease in effective stress, a reduction in the interparticle bonds, and a rearrangement of the soil particles. An ‘‘undisturbed’’ soil specimen will have little rearrangement of the soil particles and perhaps no disturbance except that caused by stress relief where there is a change from the in-situ stress condition to an isotropic ‘‘perfect sample’’ stress condition. A disturbed soil specimen will have a disrupted soil structure with perhaps a total rearrangement of soil particles. When measuring the shear strength or deformation characteristics of the soil, the results of laboratory tests run on undisturbed specimens obviously better represent in-situ properties than laboratory tests run on disturbed specimens.

Soil samples recovered from the borehole should be kept within the sampling tube or sampling rings. The soil sampling tube should be tightly sealed with end caps or the sampling rings thoroughly sealed in containers to prevent a loss of moisture during transportation to the laboratory. The soil samples should be marked with the file or project number, date of sampling, name of engineer or geologist who performed the sampling, and boring number and depth.

6.2.4 Field Testing

There are many different types of tests that can be performed at the time of drilling. The three most common types of field tests are discussed in this section:

Standard Penetration Test (SPT). The Standard Penetration Test (SPT) consists of driving a thick-walled sampler into a sand deposit. The SPT sampler must have an inside barrel diameter (D_i) = 3.81 cm (1.5 in) and an outside diameter (D_o) = 5.08 cm (2 in). The SPT sampler is shown in Fig. 6.1. The SPT sampler is driven into the sand by using a 63.5-kg (140-lb.) hammer falling a distance of 0.76 m (30 in). The SPT sampler is driven a total of 45 cm (18 in), with the number of blows recorded for each 15 cm (6 in) interval. The ''measured SPT N value'' (blows per ft) is defined as the penetration resistance of the sand, which equals the sum of the number of blows required to drive the SPT sampler over the depth interval of 15 to 45 cm (6 to 18 in). The reason the number of blows required to drive the SPT sampler for the first 15 cm (6 in) is not included in the N value is that the drilling process often disturbs the soil at the bottom of the borehole and the readings at 15 to 45 cm (6 to 18 in) are believed to be more representative of the in-situ penetration resistance of the sand. The data below present a correlation between the measured SPT N value (blows per ft) and the density condition of a clean sand deposit.

N value (blows per ft)	Sand density	Relative density
0 to 4	Very loose condition	0 to 15%
4 to 10	Loose condition	15 to 35%
10 to 30	Medium condition	35 to 65%
30 to 50	Dense condition	65 to 85%
Over 50	Very dense condition	85 to 100%

Relative density is defined in Art. 6.3.4. Note that the above correlation is very approximate and the boundaries between different density conditions are not as distinct as implied by the table.

The measured SPT N value can be influenced by many testing factors and soil conditions. For example, gravel-size particles increase the driving resistance (hence increased N value) by becoming stuck in the SPT sampler tip or barrel. Another factor that could influence the measured SPT N value is groundwater. It is important to maintain a level of water in the borehole at or above the in-situ groundwater level. This is to prevent groundwater from rushing into the bottom of the borehole, which could loosen the sand and result in low measured N values.

Besides gravel and groundwater conditions described above, there are many different testing factors that can influence the accuracy of the SPT readings. For example, the measured SPT N value could be influenced by the hammer efficiency, rate at which the blows are applied, borehole diameter, and rod lengths. The following equation is used to compensate for these testing factors (A. W. Skempton, ''Standard Penetration Test Procedures,'' *Geotechnique* 36):

$$N_{60} = 1.67\ E_m C_b C_r\ N \tag{6.3}$$

where N_{60} = SPT N value corrected for field testing procedures.
E_m = hammer efficiency (for U.S. equipment, E_m equals 0.6 for a safety hammer and E_m equals 0.45 for a donut hammer)
C_b = borehole diameter correction (C_b = 1.0 for boreholes of 65 to 115 mm (2.5 to 4.5 in) diameter, 1.05 for 150-mm diameter (5.9-in), and 1.15 for 200-mm (7.9-in) diameter hole)
C_r = Rod length correction (C_r = 0.75 for up to 4 m (13 ft) of drill rods, 0.85 for 4 to 6 m (13 to 20 ft) of drill rods, 0.95 for 6 to 10 m (20 to 33 ft) of drill rods, and 1.00 for drill rods in excess of 10 m (33 ft)
N = measured SPT N value

Even with the limitations and all of the corrections that must be applied to the measured SPT N value, the Standard Penetration Test is probably the most widely used field test in the United States. This is because it is relatively easy to use, the test is economical as compared to other types of field testing, and the SPT equipment can be quickly adapted and included as part of almost any type of drilling rig.

Cone Penetration Test (CPT). The idea for the Cone Penetration Test (CPT) is similar to that for the Standard Penetration Test, except that instead of a thick-walled sampler being driven into the soil, a steel cone is pushed into the soil. There are many different types of cone penetration devices, such as the mechanical cone, mechanical-friction cone, electric cone, and piezocone. The simplest type of cone is shown in Fig. 6.2. The cone is first pushed into the soil to the desired depth (initial position) and then a force is applied to the inner rods that moves the cone downward into the extended position. The force required to move the cone into the extended position (Fig. 6.2) divided by the horizontally projected area of the cone is defined as the cone resistance (q_c). By continual repetition of the two-step process shown in Fig. 6.2, the cone resistance data is obtained at increments of depth. A continuous record of the cone resistance versus depth can be obtained by using the electric cone, where the cone is pushed into the soil at a rate of 10 to 20 mm/sec (2 to 4 ft/min). Figure 6.3 presents four simplified examples of cone resistance (q_c) versus depth profiles and the possible interpretation of the soil types and conditions.

A major advantage of the Cone Penetration Test is that by use of the electric cone, a continuous subsurface record of the cone resistance (q_c) can be obtained. This is in contrast to the Standard Penetration Test, which obtains data at intervals in the soil deposit. Disadvantages of the Cone Penetration Test are that soil samples can not be recovered and special equipment is required to produce a steady and slow penetration of the cone. Unlike the SPT, the ability to obtain a steady and slow penetration of the cone is not included as part of conventional drilling rigs. Because of these factors, in the United States the CPT is used less frequently than the SPT.

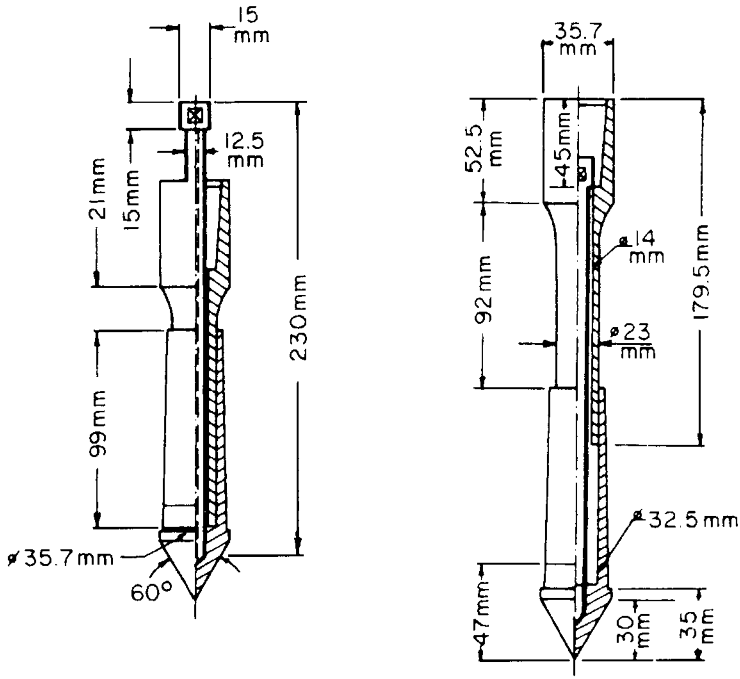


FIGURE 6.2 Example of Mechanical Cone Penetrometer Tip (Dutch Mantle Cone). (*Reprinted with permission from the American Society for Testing and Materials, 1998.*)

Vane Shear Test (VST). The SPT and CPT are used to correlate the resistance of driving a sampler (N value) or pushing a cone (q_c) with the engineering properties (such as density condition) of the soil. In contrast, the Vane Test is a different in-situ field test because it directly measures a specific soil property, the undrained shear strength (s_u) of clay. Shear strength will be further discussed in Art. 6.3.6.

The Vane Test consists of inserting a four-bladed vane, such as shown in Fig. 6.4, into the borehole and then pushing the vane into the clay deposit located at the bottom of the borehole. Once the vane is inserted into the clay, the maximum torque (T_{max}) required to rotate the vane and shear the clay is measured. The undrained shear strength (s_u) of the clay can then be calculated by using the following equation, which assumes uniform end shear for a rectangular vane:

$$s_u = \frac{T_{\max}}{\pi(0.5\, D^2H + 0.167D^3)} \tag{6.4}$$

where T_{max} = maximum torque required to rotate the rod which shears the clay
H = height of the vane
D = diameter of the vane

The vane can provide an undrained shear strength (s_u) that is too high if the vane is rotated too rapidly. The vane test also gives unreliable results for clay strata that

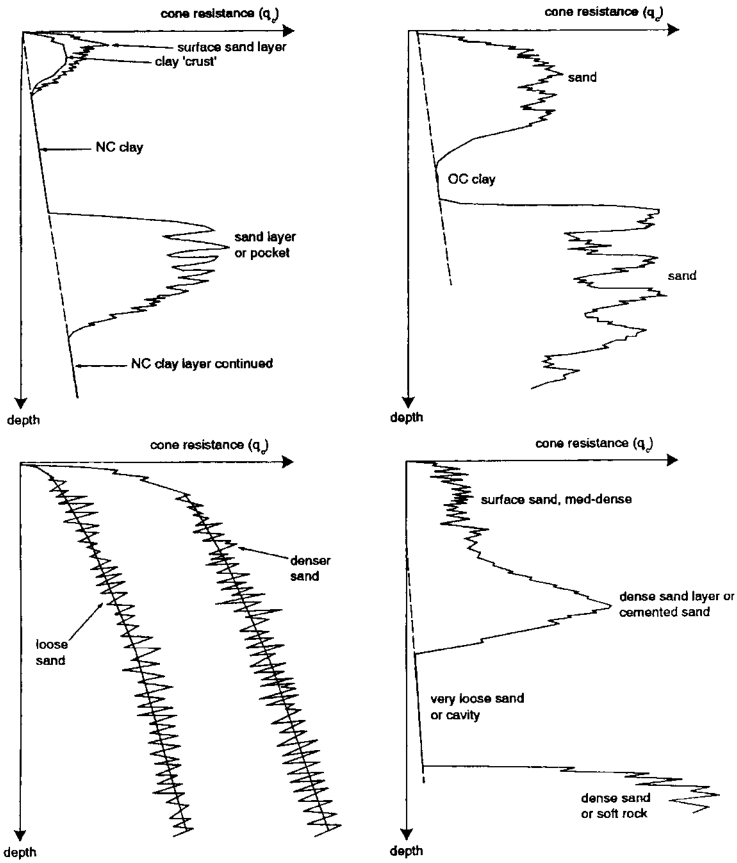


FIGURE 6.3 Simplified examples of CPT cone resistance q_c versus depth showing possible interpretations of soil types and conditions. (*From J. H. Schmertmann, "Guidelines for Cone Penetration Test." U.S. Department of Transportation, Washington, DC.*)

contains sand layers or lenses, varved clay, or if the clay contains gravel or gravel-size shell fragments.

6.2.5 Exploratory Logs

A log is defined as a written record, prepared during the subsurface excavation of borings, test pits, or trenches, that documents the observed conditions. Although logs are often prepared by technicians or even the driller, the most appropriate individuals to log the subsurface conditions are geotechnical engineers or engineering geologists who have considerable experience and judgment acquired by many years of field practice. It is especially important that the subsurface conditions likely to have the most impact on the proposed project be adequately described. Figure 6.5 presents an example of a boring log.

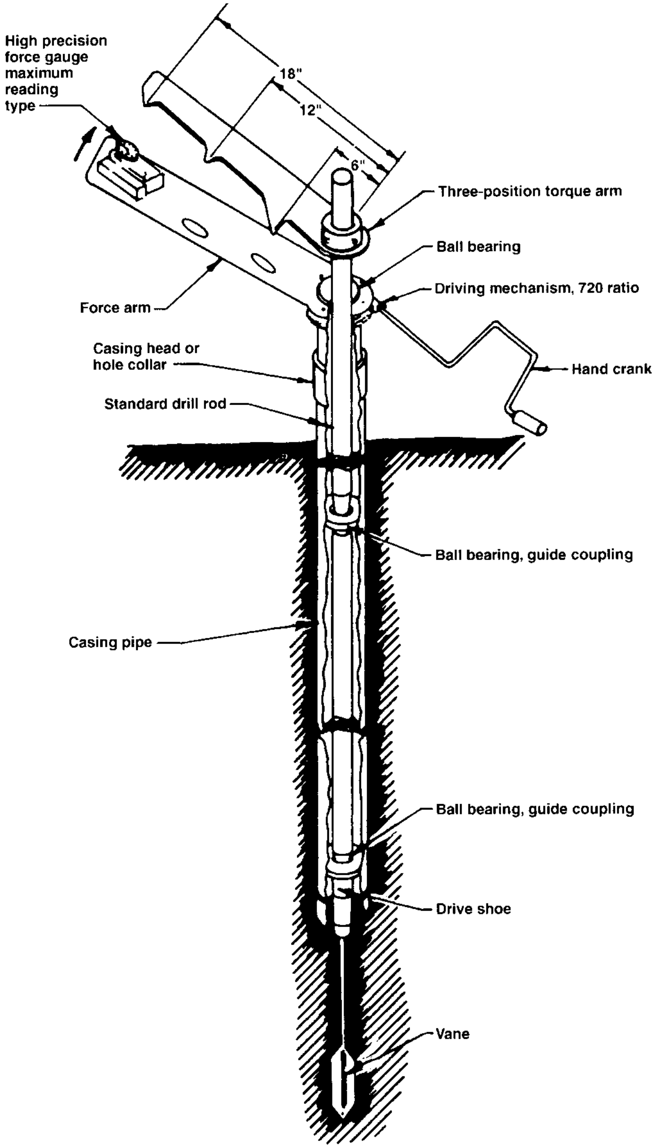


FIGURE 6.4 Diagram illustrating the Field Vane Test. (*From NAVFAC DM-7.1, 1982.*)

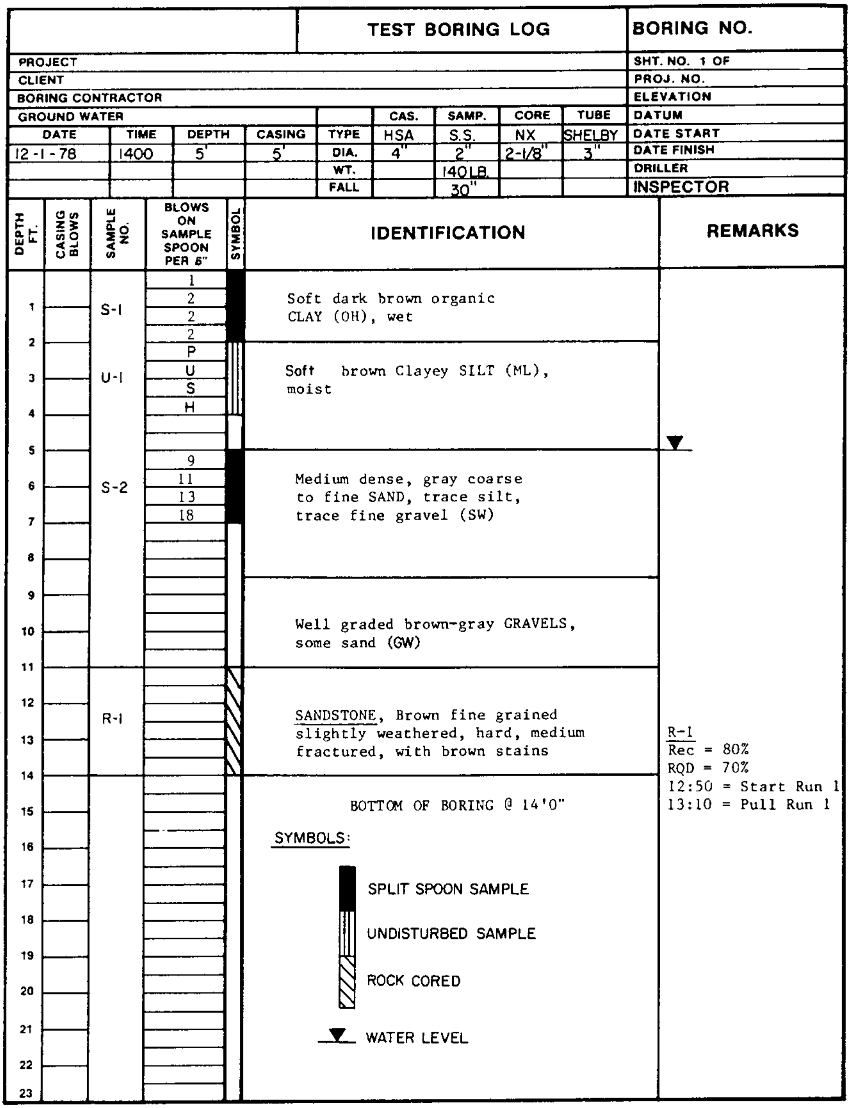


FIGURE 6.5 Example of a Boring log. (*Reproduced from NAVFAC DM-7.1, 1982.*)

6.2.6 Subsoil Profile

The final part of Art. 6.2 presents an example of a subsoil profile. As shown in Figure 6.6, the subsoil profile summarizes the results of the subsurface exploration. The results of field and laboratory tests are often included on the subsoil profile. The development of a subsoil profile is often a required element for geotechnical and foundation engineering analyses. For example, subsoil profiles are used to de-

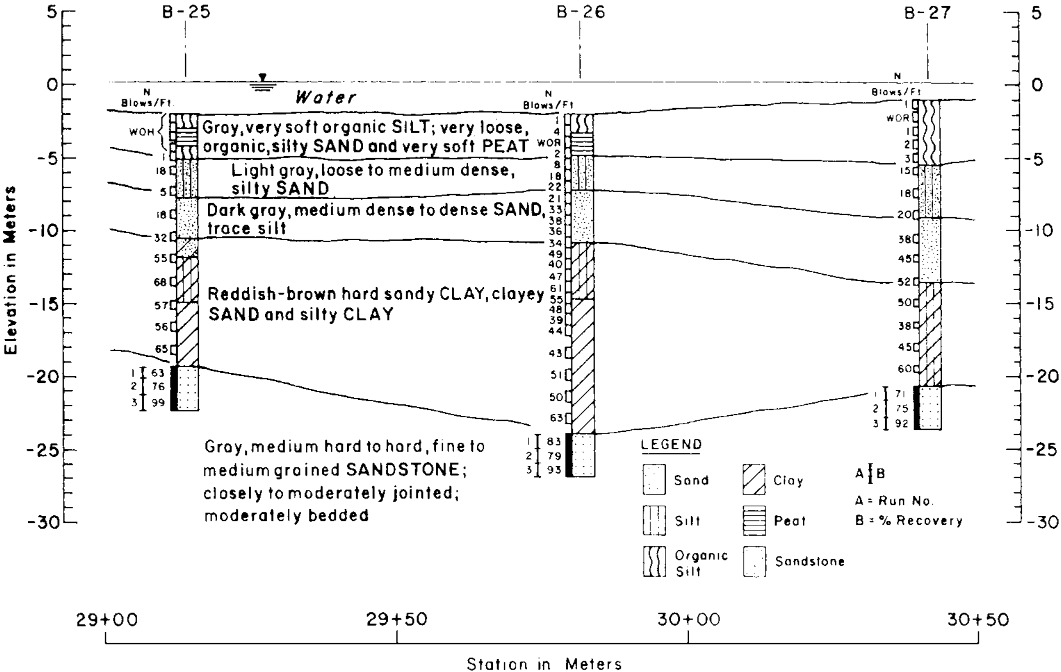


FIGURE 6.6 Subsoil profile. (*From J. Lowe and P. F. Zaccheo, "Subsurface Explorations and Sampling," ch. 1 of "Foundation Engineering Handbook," ed. H. F. Winterkorn and H.-Y. Fang, Van Nostrand Reinhold Co., New York.*)

termine the foundation type (shallow versus deep foundation), calculate the amount of settlement of the structure, evaluate the effect of groundwater on the project and develop recommendations for dewatering of underground structures, perform slope stability analyses for projects having sloping topography, and prepare site development recommendations.

6.3 LABORATORY TESTING

6.3.1 Introduction

In addition to document review and subsurface exploration, an important part of the site investigation is laboratory testing. The laboratory testing usually begins once the subsurface exploration is complete. The first step in the laboratory testing is to log in all of the materials (soil, rock, or groundwater) recovered from the subsurface exploration. Then the geotechnical engineer and engineering geologist prepare a laboratory testing program, which basically consists of assigning specific laboratory tests for the soil specimens. The actual laboratory testing of the soil specimens is often performed by experienced technicians, who are under the supervision of the geotechnical engineer. Because the soil samples can dry out or changes in the soil structure could occur with time, it is important to perform the laboratory tests as soon as possible.

Usually at the time of the laboratory testing, the geotechnical engineer and engineering geologist will have located the critical soil layers or subsurface conditions that will have the most impact on the design and construction of the project. The laboratory testing program should be oriented towards the testing of those critical soil layers or subsurface conditions. For many geotechnical projects, it is also important to determine the amount of ground surface movement due to construction of the project. In these cases, laboratory testing should model future expected conditions so that the amount of movement or stability of the ground can be analyzed.

Laboratory tests should be performed in accordance with standard procedures, such as those recommended by the American Society for Testing and Materials (ASTM) or those procedures listed in standard textbooks or specification manuals.

For laboratory tests, it has been stated (M. J. Tomlinson, ‘‘Foundation Design and Construction,’’ 5th ed., John Wiley & Sons, Inc., New York):

> It is important to keep in mind that natural soil deposits are variable in composition and state of consolidation; therefore it is necessary to use considerable judgment based on common sense and practical experience in assessing test results and knowing where reliance can be placed on the data and when they should be discarded. It is dangerous to put blind faith in laboratory tests, especially when they are few in number. The test data should be studied in conjunction with the borehole records and the site observations, and any estimations of bearing pressures or other engineering design data obtained from them should be checked as far as possible with known conditions and past experience. Laboratory tests should be as simple as possible. Tests using elaborate equipment are time-consuming and therefore costly, and are liable to serious error unless carefully and conscientiously carried out by highly experienced technicians. Such methods may be quite unjustified if the samples are few in number, or if the cost is high in relation to the cost of the project. Elaborate and costly tests are justified only if the increased accuracy of the data will give worthwhile savings in design or will eliminate the risk of a costly failure.

6.3.2 Soil Element

In order to analyze the results of laboratory tests, the concept of the soil element must be introduced. Figure 6.7 shows an element of soil that can be divided into three basic parts:

1. Solids—the mineral soil particles
2. Liquids—usually water that is contained in the void spaces between the solid mineral particles
3. Gas—such as air that is also contained in the void spaces between the solid mineral particles

As indicated on the right side of Fig. 6.7, the three basic parts of soil can be rearranged into their relative proportions based on volume and mass. Note that the symbols as defined in Fig. 6.7 will be used throughout this section.

6.3.3 Index Tests

Index tests are the most basic types of laboratory tests performed on soil samples. Index tests include the water content (also known as moisture content), specific gravity tests, unit weight determinations, and particle size distributions and Atterberg limits, which are used to classify the soil.

Water Content **(*w*).** The water content (also known as moisture content) test is probably the most common and simplest type of laboratory test. This test can be performed on disturbed or undisturbed soil specimens. The water content test consists of determining the mass of the wet soil specimen and then drying the soil in an oven overnight (12 to 16 hr) at a temperature of 110°C (ASTM D 2216-92, 1998). The water content (w) of a soil is defined as the mass of water in the soil (M_w) divided by the dry mass of the soil (M_s), expressed as a percentage (i.e., $w = 100\ M_w/M_s$).

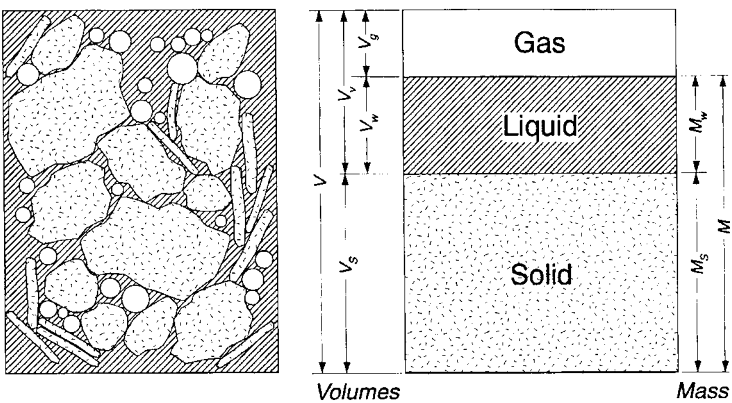


FIGURE 6.7 Soil element and the soil element separated into phases.

Values of water content (w) can vary from essentially 0% up to 1200%. A water content of 0% indicates a dry soil. An example of a dry soil would be near-surface rubble, gravel, or clean sand located in a hot and dry climate, such as Death Valley, California. Soil having the highest water content is organic soil, such as fibrous peat, which has been reported to have a water content as high as 1200%.

Specific Gravity of Soil Solids **(G).** The specific gravity (G) is a dimensionless parameter that is defined as the density of solids (ρ_s) divided by the density of water (ρ_w), or $G = \rho_s/\rho_w$. The density of solids (ρ_s) is defined as the mass of solids (M_s) divided by the volume of solids (V_s). The density of water (ρ_w) is equal to 1 g/cm^3 (or 1 Mg/m^3) and 62.4 pcf.

For soil, the specific gravity is obtained by measuring the dry mass of the soil and then using a pycnometer to obtain the volume of the soil. Table 6.5 presents typical values and ranges of specific gravity versus different types of soil minerals. Because quartz is the most abundant type of soil mineral, the specific gravity for inorganic soil is often assumed to be 2.65. For clays, the specific gravity is often assumed to be 2.70 because common clay particles, such as montmorillonite and illite, have slightly higher specific gravity values.

Total Unit Weight **(γ_t).** The total unit weight (also known as the wet unit weight) should only be obtained from undisturbed soil specimens, such as those extruded from Shelby tubes or on undisturbed block samples obtained from test pits and trenches. The first step in the laboratory testing is to determine the wet density, defined as $\rho_t = M/V$, where M = total mass of the soil, which is the sum of the mass of water (M_w) and mass of solids (M_s), and V = total volume of the soil

TABLE 6.5 Formula and Specific Gravity of Common Soil Minerals

Type of mineral	Formula	Specific gravity	Comments
Quartz	SiO_2	2.65	Silicate, most common type of soil mineral
K Feldspar	$KAlSi_3O_8$	2.54–2.57	Feldspars are also silicates and are the second most common type of soil mineral.
Na or Ca Feldspar	$NaAlSi_3O_8$	2.62–2.76	
Calcite	$CaCO_3$	2.71	Basic constituent of carbonate rocks
Dolomite	$CaMg(CO_3)_2$	2.85	Basic constituent of carbonate rocks
Muscovite	varies	2.76–3.0	Silicate sheet type mineral (mica group)
Biotite	complex	2.8–3.2	Silicate sheet type mineral (mica group)
Hematite	Fe_2O_3	5.2–5.3	Frequent cause of reddish-brown color in soil
Gypsum	$CaSO_4{\cdot}2H_2O$	2.35	Can lead to sulfate attack of concrete
Serpentine	$Mg_3Si_2O_5(OH)_4$	2.5–2.6	Silicate sheet or fibrous type mineral
Kaolinite	$Al_2Si_2O_5(OH)_4$	2.61–2.66	Silicate clay mineral, low activity
Illite	complex	2.60–2.86	Silicate clay mineral, intermediate activity
Montmorillonite	complex	2.74–2.78	Silicate clay mineral, highest activity

NOTE: Silicates are very common and account for about 80% of the minerals at the Earth's surface.

sample as defined in Fig. 6.7. The volume (V) is determined by trimming the soil specimen to a specific size or extruding the soil specimen directly from the sampler into confining rings of known volume, and then the total mass (M) of the soil specimen is obtained by using a balance.

The next step is to convert the wet density (ρ_t) to total unit weight (γ_t). In order to convert wet density to total unit weight in the International System of Units (SI), the wet density is multiplied by g (where g = acceleration of gravity = 9.81 m/sec^2) to obtain the total unit weight, which has units of kN/m^3. For example, in the International System of Units, the density of water (ρ_w) = 1.0 g/cm^3 or 1.0 Mg/m^3, while the unit weight of water (γ_w) = 9.81 kN/m^3.

In the United States Customary System, density and unit weight have exactly the same value. Thus, the density of water and the unit weight of water are 62.4 pcf. However, for the density of water (ρ_w), the units should be thought of as lb-mass (lbm) per cubic ft, while for unit weight (γ_w), the units are lb-force (lbf) per cubic foot. In the United States Customary System, it is common to assume that 1 lbm = 1 lbf.

Typical values for total unit weight (γ_t) are 110 to 130 pcf (17 to 20 kN/m^3). Besides the total unit weight, other types of unit weight are used in geotechnical engineering. For example, the dry unit weight (γ_d) refers to only the dry soil per volume, while the saturated unit weight (γ_{sat}) refers to a special case where all the soil voids are filled with water (i.e., saturated soil). Another commonly used unit weight is the buoyant unit weight (γ_b) which is used for calculations involving soil located below the groundwater table. Table 6.6 presents various equations used to

TABLE 6.6 Unit Weight Relationships*

Parameter	Relationships
Total unit weight (γ_t)	$\gamma_t = \dfrac{W_s + W_w}{V} = \dfrac{G\gamma_w(1+w)}{1+e}$
Dry unit weight (γ_d)	$\gamma_d = \dfrac{W_s}{V} = \dfrac{G\gamma_w}{1+e} = \dfrac{\gamma_t}{1+w}$
Saturated unit weight (γ_{sat})	$\gamma_{sat} = \dfrac{W_s + V_v\gamma_w}{V} = \dfrac{(G+e)\gamma_w}{1+e} = \dfrac{G\gamma_w(1+w)}{1+Gw}$ Note: The total unit weight (γ_t) is equal to the saturated unit weight (γ_{sat}) when all the void spaces are filled with water (i.e., S = 100%).
Buoyant unit weight (γ_b)	$\gamma_b = \gamma_{sat} - \gamma_w$ $\gamma_b = \dfrac{\gamma_w(G-1)}{1+e} = \dfrac{\gamma_w(G-1)}{1+Gw}$ Note: The buoyant unit weight is also known as the submerged unit weight.

*See Fig. 6.7 for definition of terms.

Notes:

1. For the equations listed in this table, water content (w) and degree of saturation (S) must be expressed as a decimal (not as a percentage).
2. ρ_w = density of water (1.0 Mg/m^3, 62.4 pcf) and γ_w = unit weight of water (9.81 kN/m^3, 62.4 pcf).

calculate the different types of unit weights. Note in Table 6.6 that w = water content and G = specific gravity of soil solids. The void ratio (e) and degree of saturation (S) are discussed in the next article.

6.3.4 Phase Relationships

Phase relationships are the basic soil relationships used in geotechnical engineering. They are also known as weight-volume relationships. Different types of phase relationships are discussed below:

***Void Ratio* (e) *and Porosity* (n).** The void ratio (e) is defined as the volume of voids (V_v) divided by the volume of solids (V_s). The porosity (n) is defined as volume of voids (V_v) divided by the total volume (V). As indicated in Fig. 6.7, the volume of voids is defined as the sum of the volume of air and volume of water in the soil.

The void ratio (e) and porosity (n) are related as follows:

$$e = \frac{n}{1 - n} \qquad \text{and} \qquad n = \frac{e}{1 + e} \tag{6.5}$$

The void ratio and porosity indicate the relative amount of void space in a soil. The lower the void ratio and porosity, the denser the soil (and vice versa). The natural soil having the lowest void ratio is probably till. For example, a typical value of dry density for till is 2.34 Mg/m^3 (146 pcf), which corresponds to a void ratio of 0.14. A typical till consists of a well-graded soil ranging in particle sizes from clay to gravel and boulders. The high density and low void ratio are due to the extremely high stress exerted by glaciers. For compacted soil, the soil type with typically the lowest void ratio is a well-graded decomposed granite (DG). A typical value of maximum dry density (Modified Proctor) for a well-graded DG is 2.20 Mg/m^3 (137 pcf), which corresponds to a void ratio of 0.21. In general, the factors needed for a very low void ratio for compacted and naturally deposited soil are as follows:

1. A well-graded grain-size distribution
2. A high ratio of D_{100}/D_0 (ratio of the largest and smallest grain sizes)
3. Clay particles (having low activity) to fill in the smallest void spaces
4. A process, such as compaction or the weight of glaciers, to compress the soil particles into dense arrangements

At the other extreme are clays, such as sodium montmorillonite, which at low confining pressures can have a void ratio of more than 25. Highly organic soil, such as peat, can have even higher void ratios.

***Degree of Saturation* (S).** The degree of saturation (S) is defined as:

$$S(\%) = \frac{100\ V_w}{V_v} \tag{6.6}$$

The degree of saturation indicates the degree to which the soil voids are filled with water. A totally dry soil will have a degree of saturation of 0%, while a saturated soil, such as a soil below the groundwater table, will have a degree of saturation of 100%. Typical ranges of degree of saturation versus soil condition are as follows:

Dry:	$S = 0\%$
Humid:	$S = 1$ to 25%
Damp:	$S = 26$ to 50%
Moist:	$S = 51$ to 75%
Wet:	$S = 76$ to 99%
Saturated:	$S = 100\%$

Relative Density. The relative density is a measure of the density state of a nonplastic soil. The relative density can only be used for soil that is nonplastic, such as sands and gravels. The relative density (D_r in %) is defined as:

$$D_r(\%) = 100 \frac{e_{\max} - e}{e_{\max} - e_{\min}} \tag{6.7}$$

where $e_{\max}$ = void ratio corresponding to the loosest possible state of the soil, usually obtained by pouring the soil into a mold of known volume
$e_{\min}$ = void ratio corresponding to the densest possible state of the soil, usually obtained by vibrating the soil particles into a dense state
e = the natural void ratio of the soil

The density state of the natural soil can be described as follows:

Very loose condition	$D_r = 0$ to 15%
Loose condition	$D_r = 15$ to 35%
Medium condition	$D_r = 35$ to 65%
Dense condition	$D_r = 65$ to 85%
Very dense condition	$D_r = 85$ to 100%

The relative density (D_r) should not be confused with the relative compaction (RC), which will be discussed in Art. 6.10.1.

Useful Relationships. A frequently used method of solving phase relationships is first to fill in the phase diagram shown in Fig. 6.7. Once the different mass and volumes are known, the various phase relationships can be determined. Another approach is to use equations that relate different parameters. A useful relationship is as follows:

$$Gw = Se \tag{6.8}$$

where G = specific gravity of soil solids
w = water content
S = degree of saturation
e = void ratio

Other commonly used relationships are presented in Table 6.7.

TABLE 6.7 Mass and Volume Relationships*

Parameter	Relationships
Mass	Mass of solids (M_s) $= \dfrac{M}{1 + w} = \dfrac{M_w G}{eS} = GV\rho_w(1 - n)$
	Mass of water (M_w) $= \dfrac{eM_s S}{G} = wM_s = S\rho_w V_v$
	Total mass (M) $= M_s + M_w = M_s(1 + w)$
Volume	Volume of solids (V_s) $= \dfrac{M_s}{G\,\rho_w} = \dfrac{V}{1 + e} = \dfrac{V_v}{e} = V(1 - n) = V - (V_g + V_w)$
	Volume of water (V_w) $= \dfrac{M_w}{\rho_w} = \dfrac{SVe}{1 + e} = SV_s\, e = S\, V_v = V_v - V_g$
	Volume of gas (V_g) $= \dfrac{(1 - S)Ve}{1 + e} = (1 - S)V_s e = V - (V_s + V_w) = V_v - V_w$
	Volume of voids (V_v) $= \dfrac{V_s n}{1 - n} = V - \dfrac{M_s}{G\,\rho_w} = \dfrac{Ve}{1 + e} = V_s e = V - V_s$
	Total volume (V) $= \dfrac{V_s}{1 - n} = \dfrac{V_v(1 + e)}{e} = V_s(1 + e) = V_s + V_g + V_w$

*See Fig. 6.7 for definition of terms.

6.3.5 Soil Classification

The purpose of soil classification is to provide the geotechnical engineer with a way to predict the behavior of the soil for engineering projects. There are many different soil classification systems in use, and only three of the most commonly used systems will be discussed in this section.

***Unified Soil Classification System* (*USCS*).** As indicated in Table 6.8, this classification system separates soils into two main groups: **coarse-grained soils** (more than 50% by weight of soil particles retained on No. 200 sieve) and **fine-grained soils** (50% or more by weight of soil particles pass the No. 200 sieve).

The coarse-grained soils are divided into **gravels** and **sands.** Both gravels and sands are further subdivided into four secondary groups as indicated in Table 6.8. The four secondary classifications are based on whether the soil is well graded, poorly graded, contains silt-sized particles, or contains clay-sized particles. These data are obtained from a particle size distribution, also known as a ''grain size curve,'' which is obtained from laboratory testing (sieve and hydrometer tests). Figure 6.8 presents examples of grain size curves.

The Atterberg limits are used to classify fine-grained soil, and they are defined as follows:

Liquid Limit (LL). The water content corresponding to the behavior change between the liquid and plastic state of a silt or clay. The liquid limit is deter-

TABLE 6.8 Unified Soil Classification System (USCS)

Major divisions	Subdivisions	USCS symbol	Typical names	Laboratory classification criteria	
Coarse-grained soils (More than 50% retained on No. 200 sieve)	Gravels (More than 50% of coarse fraction retained on No. 4 sieve)	GW	Well-graded gravels or gravel-sand mixtures, little or no fines	Less than 5% fines[a]	$C_u \geq 4$ and $1 \leq C_c \leq 3$
		GP	Poorly graded gravels or gravelly sands, little or no fines	Less than 5% fines[a]	Does not meet C_u and/or C_c criteria listed above
		GM	Silty gravels, gravel-sand-silt mixtures	More than 12% fines[a]	Minus No. 40 soil plots below the A-line
		GC	Clayey gravels, gravel-sand-clay mixtures	More than 12% fines[a]	Minus No. 40 soil plot on or above the A-line
	Sands (50% or more of coarse fraction passes No. 4 sieve)	SW	Well-graded sands or gravelly sands, little or no fines	Less than 5% fines[a]	$C_u \geq 6$ and $1 \leq C_c \leq 3$
		SP	Poorly graded sands or gravelly sands, little or no fines	Less than 5% fines[a]	Does not meet C_u and/or C_c criteria listed above
		SM	Silty sands, sand-silt mixtures	More than 12% fines[a]	Minus No. 40 soil plots below the A-line
		SC	Clayey sands, sand-clay mixtures	More than 12% fines[a]	Minus No. 40 soil plots on or above the A-line

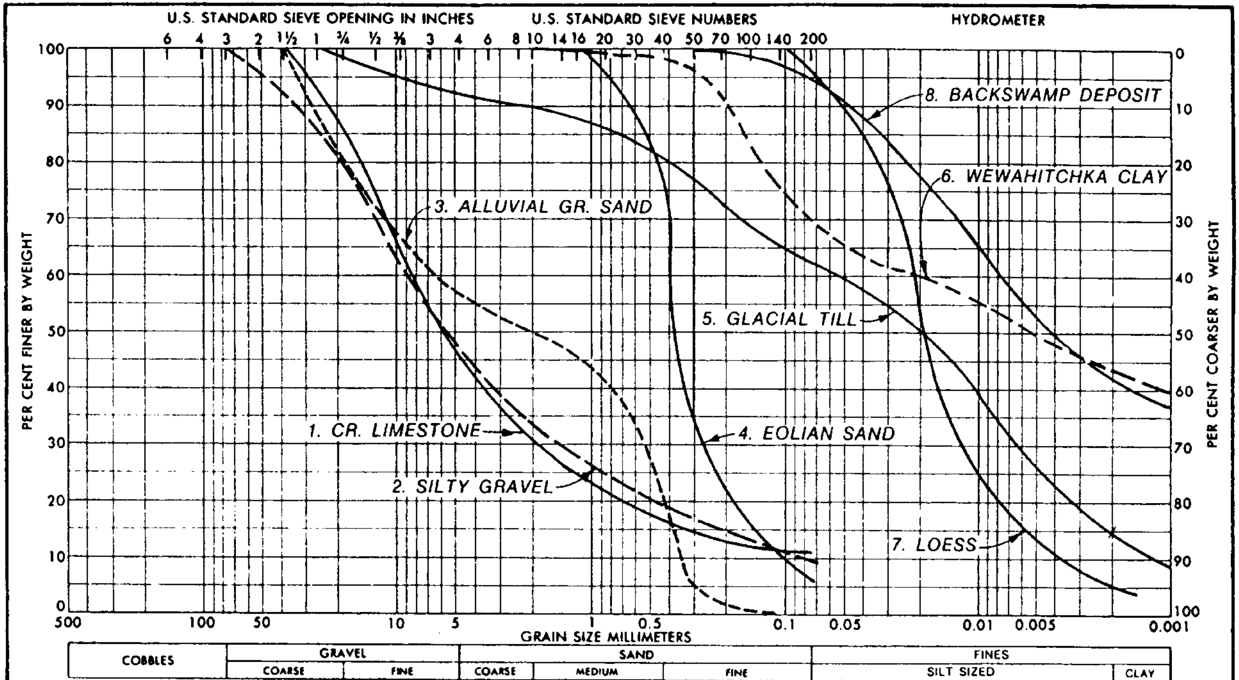
TABLE 6.8 Unified Soil Classification System (USCS) (Continued)

Major divisions	Subdivisions	USCS symbol	Typical names	Laboratory classification criteria	
Fine-grained soils (50% or more passes the No. 200 sieve)	Silts and clays (liquid limit less than 50)	ML	Inorganic silts, rock flour, silts of low plasticity	Inorganic soil	PI $<$ 4 or plots below A-line
		CL	Inorganic clays of low plasticity, gravelly clays, sandy clays, etc.	Inorganic soil	PI $>$ 7 and plots on or above A-line[b]
		OL	Organic silts and organic clays of low plasticity	Organic soil	LL (oven dried)/LL (not dried) $<$ 0.75
	Silts and clays (liquid limit 50 or more)	MH	Inorganic silts, micaceous silts, silts of high plasticity	Inorganic soil	Plots below A-line
		CH	Inorganic highly plastic clays, fat clays, silty clays, etc.	Inorganic soil	Plots on or above A-line
		OH	Organic silts and organic clays of high plasticity	Organic soil	LL (oven dried)/LL (not dried) $<$ 0.75
Peat	Highly organic	PT	Peat and other highly organic soils	Primarily organic matter, dark in color, and organic odor	

[a] "Fines" are those soil particles that pass the No. 200 sieve. For gravels with between 5% to 12% fines, use of dual symbols required (i.e., GW-GM, GW-GC, GP-GM, or GP-GC). For sands with between 5% to 12% fines, use of dual symbols required (i.e., SW-SM, SW-SC, SP-SM, or SP-SC).

[b] If $4 \leq \mathrm{PI} \leq 7$ and plots above A-line, then dual symbol (i.e., CL-ML) is required.

[c] $C_u = D_{60}/\mathrm{D}_{10}$ and $C_c = (D_{30})^2/[(D_{10})(D_{60})]$ where D_{60} = soil particle diameter corresponding to 60% finer by weight (from grain size curve).



NO.	SOIL DESCRIPTION	W_l	W_p	PI	NO.	SOIL DESCRIPTION	W_l	W_p	PI
1	CRUSHED LIMESTONE, TENNESSEE	18	11	7	5	GLACIAL TILL, ILLINOIS	25	15	10
2	SILTY GRAVEL FROM WEATHERED GABBRO, OMAN	23	20	3	6	WEWAHITCHKA SANDY CLAY, FLORIDA	65	25	41
3	ALLUVIAL GRAVELLY SAND, MISSISSIPPI	-	-	-	7	LOESS, MISSISSIPPI	29	24	5
4	EOLIAN SAND, OMAN	-	-	-	8	BACKSWAMP DEPOSIT, MISSISSIPPI RIVER	59	18	41

FIGURE 6.8 Examples of grain size curves and Atterberg limit test data for different soils. Note that w_1 = liquid limit and w_p = plastic limit. (*Reproduced from M. P. Rollings and R. S. Rollings, "Geotechnical Materials in Construction," McGraw-Hill Publishing Co., New York, with permission of McGraw-Hill, Inc.*)

mined in the laboratory by using a liquid limit device. The liquid limit is defined as the water content at which a pat of soil, cut by a groove of standard dimensions, will flow together for a distance of 12.7 mm (0.5 in) under the impact of 25 blows in a standard liquid limit device.

Plastic Limit (PL). The water content corresponding to the behavior change between the plastic and semisolid state of a silt or clay. The plastic limit is also determined in the laboratory and is defined as the water content at which a silt or clay will just begin to crumble when rolled into a tread approximately 3.2 mm (0.125 in) in diameter.

The plasticity index (PI) is defined as the liquid limit minus the plastic limit (i.e., PI = LL − PL). With both the liquid limit and plasticity index of the fine-grain soil known, the plasticity chart (Fig. 6.9) is then used to classify the soil. There are three basic dividing lines on the plasticity chart, the LL = 50 line, the A-line, and the U-line. The LL = 50 line separates soils into high and low plasticity, the A-line separates clays from silts, and the U-line represents the upper-limit line (i.e., uppermost boundary of test data).

As indicated in Table 6.8, symbols (known as "group symbols") are used to identify different soil types. The group symbols consist of two capital letters. The first letter indicates the following: G for gravel, S for sand, M for silt, C for clay, and O for organic. The second letter indicates the following: W for well graded, which indicates that a coarse-grained soil has particles of all sizes; P for poorly graded, which indicates that a coarse-grained soil has particles of the same size, or the soil is skip-graded or gap-graded; M for a coarse-grained soil that has silt-sized particles; C for a coarse-grained soil that has clay-sized particles; L for a fine-grained soil of low plasticity; and H for a fine-grained soil of high plasticity. An exception is peat, where the group symbol is PT. Also note in Table 6.8 that certain soils require the use of dual symbols.

AASHTO Soil Classification System. This classification system was developed by the American Association of State Highway and Transportation Officials (see Table 6.9). Inorganic soils are divided into 7 groups (A-1 through A-7), with the

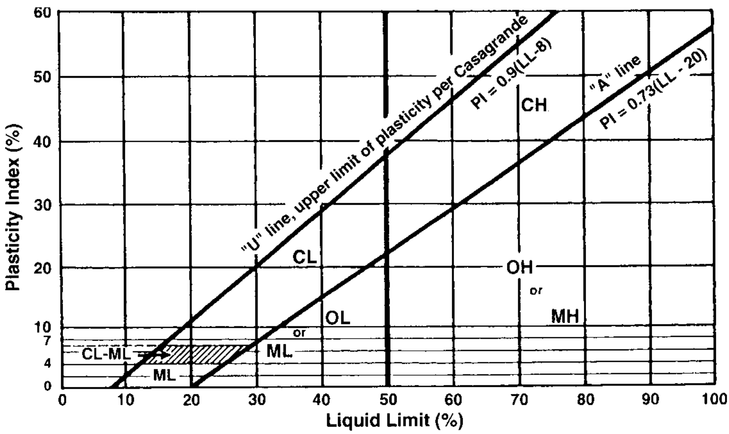


FIGURE 6.9 Plasticity chart.

TABLE 6.9 AASHTO Soil Classification System

Major Divisions	Group	AASHTO symbol	Typical names	Sieve analysis (percent passing)	Atterberg limits
Granual materials (35% or less passing No. 200 sieve)	Group A-1	A-1-a	Stone or gravel fragments	Percent Passing: No. 10 $\leq$ 50% No. 40 $\leq$ 30% No. 200 $\leq$ 15%	PI $\leq$ 6
		A-1-b	Gravel and sand mixtures	No. 40 $\leq$ 50% No. 200 $\leq$ 25%	PI $\leq$ 6
	Group A-3	A-3	Fine sand that is nonplastic	No. 40 $>$ 50% No. 200 $\leq$ 10%	PI = 0 (nonplastic)
	Group A-2	A-2-4	Silty gravel and sand	Percent passing No. 200 sieve $\leq$ 35%	LL $\leq$ 40 PI $\leq$ 10
		A-2-5	Silty gravel and sand	Percent passing No. 200 sieve $\leq$ 35%	LL $>$ 40 PI $\leq$ 10
		A-2-6	Clayey gravel and sand	Percent passing No. 200 sieve $\leq$ 35%	LL $\leq$ 40 PI $>$ 10
		A-2-7	Clayey gravel and sand	Percent passing No. 200 sieve $\leq$ 35%	LL $>$ 40 PI $>$ 10
Silt-clay materials (More than 35% passing No. 200 sieve)	Group A-4	A-4	Silty soils	Percent passing No. 200 sieve $>$ 35%	LL $\leq$ 40 PI $\leq$ 10
	Group A-5	A-5	Silty soils	Percent passing No. 200 sieve $>$ 35%	LL $>$ 40 PI $\leq$ 10
	Group A-6	A-6	Clayey soils	Percent passing No. 200 sieve $>$ 35%	LL $\leq$ 40 PI $>$ 10
	Group A-7	A-7-5	Clayey soils	Percent passing No. 200 sieve $>$ 35%	LL $>$ 40 PI $\leq$ LL $-$ 30 PI $>$ 10
		A-7-6	Clayey soils	Percent passing No. 200 sieve $>$ 35%	LL $>$ 40 PI $>$ LL $-$ 30 PI $>$ 10
Highly organic	Group A-8	A-8	Peat and other highly organic soils	Primarily organic matter, dark in color, and organic odor	

Notes:

1. Classification Procedure: First decide which of the three main categories (granular materials, silt-clay materials, or highly organic) the soil belongs. Then proceed from the top to the bottom of the chart and the first group that meets the particle size and Atterberg limits criteria is the correct classification.
2. Group Index $= (F - 35)[0.2 + 0.005(LL - 40)] + 0.01(F - 15)(PI - 10)$, where F = percent passing No. 200 sieve, LL = liquid limit, and PI = plasticity index. Report group index to nearest whole number. For negative group index, report as zero. When working with A-2-6 and A-2-7 subgroups, use only the PI portion of the group index equation.
3. Atterberg limits are performed on soil passing the No. 40 sieve. LL = liquid limit, PL = plastic limit, and PI = plasticity index.
4. AASHTO definitions of particle sizes are as follows: (a) boulders: above 75 mm, (b) gravel: 75 mm to No. 10 sieve, (c) coarse sand: No. 10 to No. 40 sieve, (d) fine sand: No. 40 to No. 200 sieve, and (e) silt-clay size particles: material passing No. 200 sieve.
5. Example: An example of an AASHTO classification for a clay is A-7-6 (30), or Group A-7, subgroup 6, group index 30.

eighth group (A-8) reserved for highly organic soils. Soil types A-1, A-2, and A-7 have subgroups as indicated in Table 6.9. Those soils having plastic fines can be further categorized by using the group index (defined in Table 6.9). Groups A-1-a, A-1-b, A-3, A-2-4, and A-2-5 should be considered to have a group index equal to zero. According to AASHTO, the road supporting characteristics of a subgrade may be assumed as an inverse ratio to its group index. Thus, a road subgrade having a group index of 0 indicates a ‘‘good’’ subgrade material that will often provide good drainage and adequate bearing when thoroughly compacted. A road subgrade material that has a group index of 20 or greater indicates a ‘‘very poor’’ subgrade material that will often be impervious and have a low bearing capacity.

Organic Soil Classification System. Table 6.10 presents a classification system for organic materials. As indicated in Table 6.10, there are four major divisions, as follows:

1. **Organic Matter.** These materials consist almost entirely of organic material. Examples include fibrous peat and fine-grained peat.
2. **Highly Organic Soils.** These soils are composed of 30 to 75% organic matter mixed with mineral soil particles. Examples include silty peat and sandy peat.
3. **Organic Soils.** These soils are composed of from 5 to 30% organic material. These soils are typically classified as organic soils of high plasticity (OH, i.e. LL $\geq$ 50) or low plasticity (OL, i.e., LL $<$ 50) and have a ratio of liquid limit (oven-dried soil) divided by liquid limit (not dried soil) that is less than 0.75 (see Table 6.8).
4. **Slightly Organic Soils.** These soils typically have less than 5% organic matter. Per the Unified Soil Classification System, they have a ratio of liquid limit (oven-dried soil) divided by liquid limit (not dried soil) that is greater than 0.75. Often a modifier, such as ‘‘slightly organic soil,’’ is used to indicate the presence of organic matter.

Also included in Table 6.10 is the typical range of laboratory test results for the four major divisions of organic material. Note in Table 6.10 that the water content (w) increases and the total unit weight (γ_t) decreases as the organic content increases. The specific gravity (G) includes the organic matter, hence the low values for highly organic material. The compression index (C_c) is discussed in Art. 6.5.6.

Other Descriptive Terminology. In addition to the classification of a soil, other items should also be included in the field or laboratory description of a soil, such as:

1. **Soil Color.** Usually the standard primary color (red, orange, yellow, etc.) of the soil is listed.
2. **Soil Texture.** The texture of a soil refers to the degree of fineness of the soil. For example, terms such as **smooth, gritty,** or **sharp** can be used to describe the texture of the soil when it is rubbed between the fingers.
3. **Clay Consistency.** For clays, the consistency (i.e., degree of firmness) should be listed. The consistency of a clay varies from ‘‘very soft’’ to ‘‘hard’’ based on the undrained shear strength of the clay (s_u). The undrained shear strength can be determined from the Unconfined Compression Test or from field or laboratory vane tests. The consistency versus undrained shear strength (s_u) is as follows:

TABLE 6.10 Soil Classification for Organic Soil

Major divisions	Organic content	USCS symbol	Typical names	Distinguishing characteristics for visual identification	Typical range of laboratory test results
Organic matter	75 to 100% Organics (Either visible or inferred)	PT	Fibrous peat (woody, mats, etc.)	Light weight and spongy. Shrinks considerably on air drying. Much water squeezes from sample.	w = 500 to 1200% γ_t = 9.4 to 11 kN/m^3 (60 to 70 pcf) G = 1.2 to 1.8 $C_c/(1 + e_o) \geq 0.40$
		PT	Fine-grained peat (amorphous)	Light weight and spongy. Shrinks considerably on air drying. Much water squeezes from sample.	w = 400 to 800% PI = 200 to 500 γ_t = 9.4 to 11 kN/m^3 (60 to 70 pcf) G = 1.2 to 1.8 $C_c/(1 + e_o) \geq 0.35$
Highly organic soils	30 to 75% Organics (Either visible or inferred)	PT	Silty peat	Relatively light weight, spongy. Shrinks on air drying. Usually can readily squeeze water from sample.	w = 250 to 500% PI = 150 to 350 γ_t = 10 to 14 kN/m^3 (65 to 90 pcf) G = 1.8 to 2.3 $C_c/(1 + e_o)$ = 0.3 to 0.4
		PT	Sandy peat	Sand fraction visible. Shrinks on air drying. Often a "gritty" texture. Usually can squeeze water from sample.	w = 100 to 400% PI = 50 to 150 γ_t = 11 to 16 kN/m^3 (70 to 100 pcf) G = 1.8 to 2.4 $C_c/(1 + e_o)$ = 0.2 to 0.3
Organic soils	5 to 30% organics (Either visible or inferred)	OH	Clayey organic Silt	Often has strong hydrogen sulfide (H_2S) odor. Medium dry strength and slow dilatency.	w = 65 to 200% PI = 50 to 150 γ_t = 11 to 16 kN/m^3 (70 to 100 pcf) G = 2.3 to 2.6 $C_c/(1 + e_o)$ = 0.2 to 0.35
		OL	Organic sand or Silt	Threads weak and friable near plastic limit, or will not roll at all. Low dry strength, medium to high dilatency.	w = 30 to 125% PI = NP to 40 γ_t = 14 to 17 kN/m^3 (90 to 110 pcf) G = 2.4 to 2.6 $C_c/(1 + e_o)$ = 0.1 to 0.25
Slightly organic soils	Less than 5% organics	Use Table 6.8	Soil with slight organic fraction	Depends on the characteristics of the inorganic fraction.	Depends on the characteristics of the inorganic fraction.

Source: NAVFAC DM-7.1, 1982, based on unpublished work by Ayers and Plum.

Notes: w = in-situ water content, PI = plasticity index, NP = nonplastic, γ_t = total unit weight, G = specific gravity (soil minerals plus organic matter), C_c = compression index, e_o = initial void ratio, and $C_c/(1 + e_o)$ = modified compression index.

Soil consistency	Undrained shear strength (kPa)	Undrained shear strength (psf)
Very soft	$s_u < 12$	$s_u < 250$
Soft	$12 \le s_u < 25$	$250 \le s_u < 500$
Medium	$25 \le s_u < 50$	$500 \le s_u < 1000$
Stiff	$50 \le s_u < 100$	$1000 \le s_u < 2000$
Very stiff	$100 \le s_u < 200$	$2000 \le s_u < 4000$
Hard	$s_u \ge 200$	$s_u \ge 4000$

4. **Sand Density Condition.** For sands, the density state of the soil varies from "very loose" to "very dense." The determination of the density condition is based on the relative density (D_r in %).
5. **Soil Moisture Condition.** The moisture condition of the soil should also be listed. Based on the degree of saturation, the moisture conditions can vary from a "dry" soil ($S = 0\%$) to a "saturated" soil ($S = 100\%$).
6. **Additional Descriptive Items.** The soil classification systems are usually only applicable for soil and rock particles passing the 75-mm (3-in) sieve. Cobbles and boulders are larger than the 75 mm (3 in), and if applicable, the words "with cobbles" or "with boulders" should be added to the soil classification. Typically, cobbles refer to particles ranging from 75 mm (3 in) to 200 mm (8 in) and boulders refer to any particle over 200 mm (8 in).

Other descriptive terminology includes the presence of rock fragments, such as "crushed shale, claystone, sandstone, siltstone, or mudstone fragments," and unusual constituents such as "shells, slag, glass fragments, and construction debris."

Soil classification examples are shown on the boring log in Fig. 6.5. Common types of soil deposits are listed in Table 6.11.

6.3.6 Shear Strength Tests

The shear strength of a soil is a basic geotechnical engineering parameter and is required for the analysis of foundations, earthwork, and slope stability problems. This is because of the nature of soil, which is composed of individual soil particles that slide (i.e., shear past each other) when the soil is loaded.

The shear strength of the soil can be determined in the field (e.g., vane shear test) or in the laboratory. Laboratory shear strength tests can generally be divided into two categories:

1. **Shear Strength Tests Based on Total Stress.** The purpose of these laboratory tests is to obtain the undrained shear strength (s_u) of the soil or the failure envelope in terms of total stresses (total cohesion, c, and total friction angle, ϕ). These types of shear strength tests are often referred to as "undrained" shear strength tests.
2. **Shear Strength Tests Based on Effective Stress.** The purpose of these laboratory tests is to obtain the effective shear strength of the soil based on the failure envelope in terms of effective stress (effective cohesion, c', and effective friction angle, ϕ'). These types of shear strength tests are often referred to as "drained" shear strength tests. The shear strength of the soil can be defined as (Mohr-Coulomb failure law):

TABLE 6.11 Common Man-made and Geologic Soil Deposits

Main category	Common types of soil deposits	Possible engineering problems
Structural fill	Dense or hard fill. Often the individual fill lifts can be identified	Upper surface of structural fill may have become loose or weathered
Uncompacted fill	Random soil deposit that can contain chunks of different types and sizes of rock fragments	Susceptible to compression and collapse
Debris fill	Contains pieces of debris, such as concrete, brick, and wood fragments	Susceptible to compression and collapse
Municipal dump	Contains debris and waste products such as household garbage or yard trimmings	Significant compression and gas from organic decomposition
Residual soil deposit	Soil deposits formed by in-place weathering of rock	Engineering properties are highly variable
Organic deposit	Examples include peat and muck which forms in bogs, marshes, and swamps	Very compressible and unsuitable for foundation support
Alluvial deposit	Soil transported and deposited by flowing water, such as streams and rivers	All types of grain sizes, loose sandy deposits susceptible to liquefaction
Aeolian deposit	Soil transported and deposited by wind. Examples include loess and dune sands	Can have unstable soil structure that may be susceptible to collapse
Glacial deposit	Soil transported and deposited by glaciers or their melt water. Examples include till.	Erratic till deposits and soft clay deposited by glacial melt water
Lacustrine deposit	Soil deposited in lakes or other inland bodies of water	Unusual soil deposits can form, such as varved silts or varved clays
Marine deposit	Soil deposited in the ocean, often from rivers that empty into the ocean	Granular shore deposits but offshore areas can contain soft clay deposits
Colluvial deposit	Soil transported and deposited by gravity, such as talus, hill-wash, or landslide deposits	Can be geologically unstable deposit
Pyroclastic deposit	Material ejected from volcanoes. Examples include ash, lapilli, and bombs	Weathering can result in plastic clay. Ash can be susceptible to erosion.

NOTE: The first four soil deposits are man-made, all others are due to geologic processes.

$$\tau_f = c' + \sigma_n' \tan \phi' \tag{6.9}$$

where τ_f = shear strength of the soil
c' = effective cohesion
σ_n' = effective normal stress on the shear surface
ϕ' = effective friction angle

The mechanisms that control the shear strength of soil are complex, but in simple terms the shear strength of soils can be divided into two broad categories: granular (nonplastic) soils and cohesive (plastic) soils.

Granular Soil. These types of soil are nonplastic and include gravels, sands, and nonplastic silt such as rock flour. A granular soil develops its shear strength as a result of the frictional and interlocking resistance between the individual soil particles. Granular soils, also known as cohesionless soils, can only be held together by confining pressures and will fall apart when the confining pressure is released (i.e., $c' = 0$). The drained shear strength (effective stress analysis) is of most importance for granular soils. The shear strength of granular soils is often measured in the direct shear apparatus, where a soil specimen is subjected to a constant vertical pressure (σ_n') while a horizontal force is applied to the top of the shear box so that the soil specimen is sheared in half along a horizontal shear surface (see Fig. 6.10). By plotting the vertical pressure (σ_n') versus shear stress at failure (τ_f), the effective friction angle (ϕ') can be obtained. Because the test specifications typically require the direct shear testing of soil in a saturated and drained state, the shear strength of the soil is expressed in terms of the effective friction angle (ϕ'). Granular soils can also be tested in a dry state, and the shear strength of the soil is then expressed in terms of the friction angle (ϕ). In a comparison of the effective friction angle (ϕ') from drained direct shear tests on saturated cohesionless soil and the friction angle (ϕ) from direct shear tests on the same soil in a dry state, it has been determined that ϕ' is only 1 to 2° lower than ϕ. This slight difference is usually ignored and the friction angle (ϕ) and effective friction angle (ϕ') are typically considered to mean the same thing for granular (nonplastic) soils.

Table 6.12 presents values of effective friction angles for different types of granular (nonplastic) soils. An exception to the values presented in Table 6.12 are granular soils that contain appreciable mica flakes. A micaceous sand will often have a high void ratio and hence little interlocking and a lower friction angle. In summary, for granular soils, $c' = 0$ and the effective friction angle (ϕ') depends on:

1. **Soil Type** (Table 6.12). Sand and gravel mixtures have a higher effective friction angle than nonplastic silts.
2. **Soil Density.** For a given granular soil, the denser the soil, the higher the effective friction angle. This is due to the interlocking of soil particles, where at a

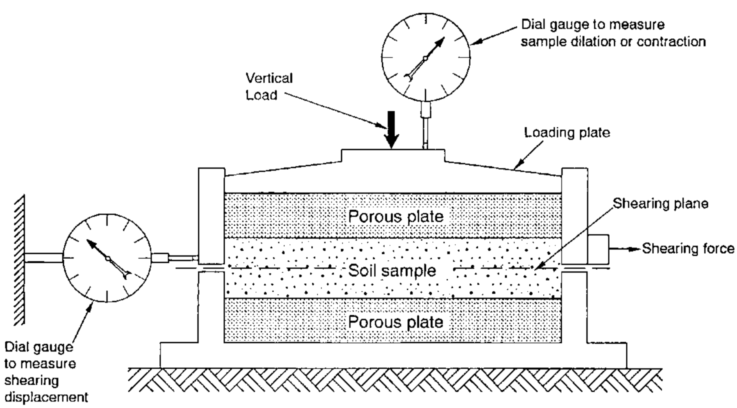


FIGURE 6.10 Direct shear apparatus.

TABLE 6.12 Typical Effective Friction Angles (ϕ') for Different Cohesionless Soils*

Soil types	Effective friction angles (ϕ') at peak strength		
	Loose	Medium	Dense
Silt (nonplastic)	26 to 30°	28 to 32°	30 to 34°
Uniform fine to medium sand	26 to 30°	30 to 34°	32 to 36°
Well-graded sand	30 to 34°	34 to 40°	38 to 46°
Sand and gravel mixtures	32 to 36°	36 to 42°	40 to 48°

*Data from B. K. Hough, "Basic Soils Engineering," 2d ed., John Wiley & Sons, Inc., New York.

denser state the soil particles are interlocked to a higher degree and hence the effective friction angle is greater than in a loose state. It has been observed that in the ultimate shear strength state, the shear strength and density of a loose and dense sand tend to approach each other.

3. **Grain Size Distribution.** A well-graded granular soil will usually have a higher friction angle than a uniform soil. With more soil particles to fill in the small spaces between soil particles, there is more interlocking and frictional resistance developed for a well-graded than a uniform granular soil.
4. **Mineral Type, Angularity, and Particle Size.** Soil particles composed of quartz tend to have a higher friction angle than soil particles composed of weak carbonate. Angular soil particles tend to have rougher surfaces and better interlocking ability. Larger-sized particles, such as gravel-sized particles, typically have higher friction angles than sand.
5. **Deposit Variability.** Because of variations in soil types, gradations, particle arrangements, and dry density values, the effective friction angle is rarely uniform with depth. It takes considerable judgment and experience in selecting an effective friction angle based on an analysis of laboratory data.
6. **Indirect Methods.** For many projects, the effective friction angle of the sand is determined by indirect means, such as the Standard Penetration Test and the Cone Penetration Test.

Cohesive Soil. The shear strength of cohesive (plastic) soil, such as silts and clays, is much more complicated than the shear strength of granular soils. Also, in general the shear strength of cohesive (plastic) soils tends to be lower than the shear strength of granular soils. As a result, more shear-induced failures occur in cohesive soils, such as clays, than in granular (nonplastic) soils.

Depending on the type of loading condition, either a total stress analysis or an effective stress analysis could be performed for cohesive soil. In general, total stress analysis (s_u or c and ϕ) are used for short-term conditions, such as at the end of construction. The total stress parameters, such as the undrained shear strength (s_u), can be determined from an unconfined compression test or vane tests.

Figure 6.11 presents an example of the undrained shear strength (s_u) versus depth for Borings E1 and F1 excavated in an offshore deposit of Orinoco clay (created by sediments from the Orinoco River, Venezuela). The Orinoco clay can be generally classified as a clay of high plasticity (CH) and can be considered to

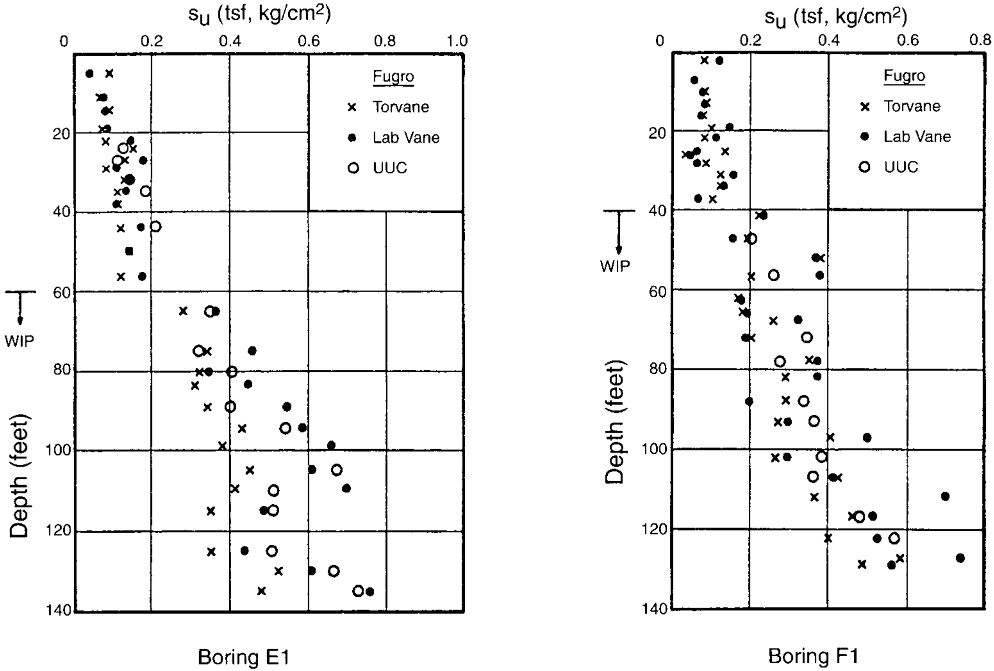


FIGURE 6.11 Undrained shear strength versus depth for Orinoco clay at Borings E1 and F1.

be a relatively uniform soil deposit. The undrained shear strength was obtained from the Torvane device, laboratory vane, and unconfined compression test (UUC). Note in Fig. 6.11 that there is a distinct discontinuity in the undrained shear strength (s_u) at a depth of 60 ft for Boring E1 and 40 ft for Boring F1. This discontinuity was due to different sampling procedures. Above a depth of 60 ft at Boring E1 and 40 ft at Boring F1, samplers were hammered into the clay deposit, causing sample disturbance and a lower shear strength value for the upper zone of clay. For the deeper zone of clay, a ''WIP'' sampling procedure was utilized, which produced less sample disturbance and hence a higher undrained shear strength.

Effective stress analyses (c' and ϕ') are used for long-term conditions, where the soil and groundwater conditions are relatively constant. Effective shear strength parameters are often obtained from laboratory triaxial tests, where a saturated soil specimen is sheared by applying a load to the top of the specimen (see Fig. 6.12). During shearing, the pore water pressures (u) are measured in order to calculate the effective friction angle of the soil. Typical values of the effective friction angle (ϕ') for natural clays range from around 20° for normally consolidated highly plastic clays up to 30° or more for other types of plastic (cohesive) soil. The value of ϕ' for compacted clay is typically in the range of 25° to 30° and occasionally as high as 35°. In terms of effective cohesion for plastic soil, the value of c' for normally consolidated noncemented clays is very small and can be assumed to be zero for practical work. These effective friction angles (ϕ') for cohesive soil are less than the values for granular soil (Table 6.12), and this is the reason there are more shear failures in cohesive than in granular soil.

6.4 EFFECTIVE STRESS AND STRESS DISTRIBUTION

It is important to recognize that without adequate and meaningful data from the field exploration (Art. 6.2) and laboratory testing (Art. 6.3), the engineering analysis presented in the rest of this chapter will be of doubtful value and may even lead to erroneous conclusions. The purpose of the engineering analysis is often to develop site development and foundation design parameters required for the project.

6.4.1 Effective Stress, Total Stress, and Pore Water Pressure

The soil has been described in terms of a written description (soil classification) and mathematical description (phase relationships). The next step in the analysis is often to determine the stresses acting on the soil. This is important because most geotechnical projects deal with a change in stress of the soil. For example, the construction of a building applies an additional stress onto the soil supporting the foundation, which results in settlement of the building.

Stress is defined as the load divided by the area over which it acts. In geotechnical engineering, a compressive stress is considered positive and tensile stress is negative. Stress and pressure are often used interchangeably in geotechnical engineering. In the International System of Units (SI), the units for stress are kPa. In the United States Customary System, the units for stress are psf (lb-force per square ft). Stress expressed in units of kg/cm^2 have been used in the past and are still in use (e.g., see Fig. 6.11). One kg/cm^2 is approximately equal to 100 kPa and one ton per square ft (tsf).

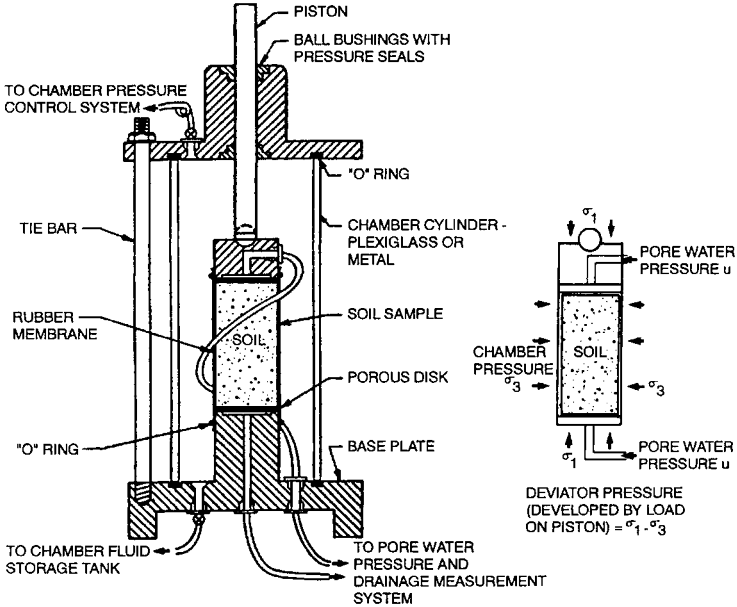


FIGURE 6.12 Triaxial apparatus.

Effective Stress. An important concept in geotechnical engineering is effective stress. The effective stress (σ') is defined as follows:

$$\sigma' = \sigma - u \tag{6.10}$$

where σ = total stress
u = pore water pressure

Many engineering analyses use the vertical effective stress, also known as the effective overburden stress, which is designated σ'_v or σ'_{vo}.

Total Stress. For the condition of a uniform soil and a level ground surface (geostatic condition), the total vertical stress (σ_v) at a depth (z) below the ground surface is:

$$\sigma_v = \gamma_t z \tag{6.11}$$

where γ_t = total unit weight of the soil (Table 6.6). For soil deposits having layers with different total unit weights, the total vertical stress is the sum of the vertical stress for each individual soil layer.

Pore Water Pressure (u) and Calculation of Vertical Effective Stress (σ'_v). For the condition of a hydrostatic groundwater table (i.e., no groundwater flow or excess pore water pressures), the static pore water pressure (u or u_s) is:

$$u = \gamma_w z_w \tag{6.12}$$

where γ_w = unit weight of water
z_w = depth below the groundwater table

If the total unit weight of the soil (Eq. 6.11), and the pore water pressure (Eq. 6.12) are known, then the vertical effective stress (σ'_v) can be calculated. An alternative method is to use the buoyant unit weight (γ_b, see Table 6.6) to calculate the vertical effective stress. For example, suppose that a groundwater table corresponds with the ground surface. In this case, the vertical effective stress (σ'_v) is simply the buoyant unit weight (γ_b) times the depth below the ground surface. More often, the groundwater table is below the ground surface, in which case the vertical total stress of the soil layer above the groundwater table must be added to the buoyant unit weight calculations.

6.4.2 Stress Distribution

The previous section described methods used to determine the existing stresses within the soil mass. This section describes commonly used methods to determine the increase in stress in the soil deposit due to applied loads. This is naturally important in settlement analysis because the settlement of the structure is due directly to its weight, which causes an increase in stress in the underlying soil. In most cases, it is the increase in vertical stress that is of most importance in settlement analyses. The symbol σ_z is often used to denote an increase in vertical stress in the soil, although $\Delta\sigma_v$ (change in total vertical stress) is also used.

When dealing with stress distribution, a distinction must be made between one-dimensional and two- or three-dimensional loading. A one-dimensional loading applies a stress increase at depth that is 100% of the applied surface stress. An example of a one-dimensional loading would be the placement of a fill layer of uniform thickness and large areal extent at ground surface. Beneath the center of the uniform fill, the in-situ soil is subjected to an increase in vertical stress that equals the following:

$$\sigma_z = \Delta\sigma_v = h\gamma_t \tag{6.13}$$

where h = thickness of the fill layer
γ_t = total unit weight of the fill

In this case of one-dimensional loading, the soil would only be compressed in the vertical direction (i.e., strain only in the vertical direction).

Another example of one-dimensional loading is the uniform lowering of a groundwater table. If the total unit weight of the soil does not change as the groundwater table is lowered, then the one-dimensional increase in vertical stress for the in-situ soil located below the groundwater table would equal the following:

$$\sigma_z = \Delta\sigma_v = h\gamma_w \tag{6.14}$$

where h = vertical distance that the groundwater table is uniformly lowered
γ_w = unit weight of water

Surface loadings can cause both vertical and horizontal strains, and this is referred to as two- or three-dimensional loading. Common examples of two-dimensional loading are from strip footings or long embankments (i.e., plane strain conditions). Examples of three-dimensional loading would be square and rectangular footings (spread footings) and round storage tanks. The following two sections describe methods that can be used to determine the change in vertical stress for two-dimensional (strip footings and long embankments) and three-dimensional (spread footings and round storage tanks) loading conditions. In these cases, the load usually dissipates rapidly with depth. The following methods will yield different answers for a given set of conditions. The reader is cautioned to follow any limitations mentioned.

2:1 Approximation. A simple method to determine the increase in vertical stress with depth is the **2:1 approximation** (also known as the **2:1 method**). Figure 6.13 illustrates the basic principle of the 2:1 approximation. This method assumes that the stress dissipates with depth in the form of a trapezoid that has 2:1 (vertical: horizontal) inclined sides as shown in Fig. 6.13. The purpose of this method is to approximate the actual "pressure bulb" stress increase beneath a footing.

If there is a strip footing of width B that has a vertical load (P) per unit length of footing, then, as indicated in Fig. 6.13, the stress applied by the footing (σ_o) would be $\sigma_o = P/B$ where B = width of the strip footing. As indicated in Fig. 6.13, at a depth z below the footing, the vertical stress increase (σ_z) due to the strip footing load would be:

$$\sigma_z = \Delta\sigma_v = \frac{P}{B + z} \tag{6.15}$$

If the footing is a rectangular spread footing having a length $= L$ and a width $= B$, then the stress applied by the rectangular footing (σ_o) would be $\sigma_o = P/(BL)$ where P = entire load of the rectangular spread footing. Based on the 2:1 approximation, the vertical stress increase (σ_z) at a depth $= z$ below the rectangular spread footing would be:

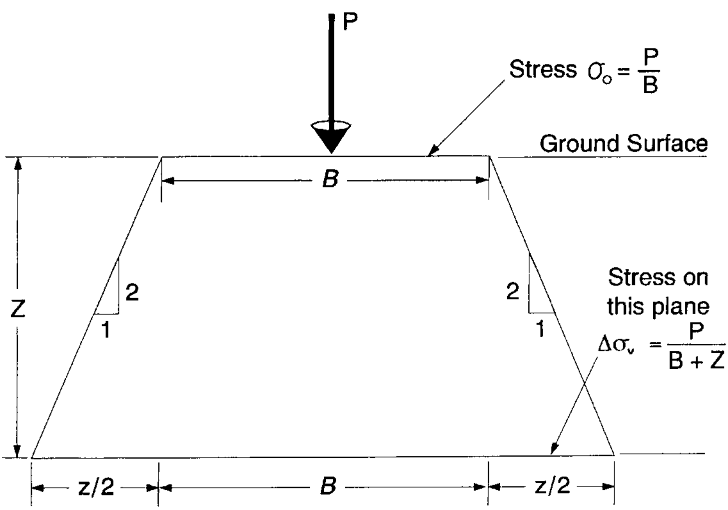


FIGURE 6.13 2:1 approximation for the calculation of the increase in vertical stress at depth due to an applied load (P).

$$\sigma_z = \Delta\sigma_v = \frac{P}{(B + z)(L + z)} \tag{6.16}$$

A major advantage of the 2:1 approximation is its simplicity, and for this reason it is probably used more often than any other type of stress distribution method. The main disadvantage with the 2:1 approximation is that the stress increase under the center of the loaded area equals the stress increase under the corner or side of the loaded area. The actual situation is that the soil underlying the center of the loaded area is subjected to a higher vertical stress increase than the soil underneath a corner or edge of the loaded area. Thus, the 2:1 approximation is often only used to estimate the average settlement of the loaded area. Different methods, such as stress distribution based on the theory of elasticity, can be used to calculate the change in vertical stress between the center and corner of the loaded area.

Equations and Charts Based on the Theory of Elasticity. Equations and charts have been developed to determine the change in stress due to applied loads based on the theory of elasticity. The solutions assume an elastic and homogeneous soil that is continuous and in static equilibrium. The elastic solutions also use a specific type of applied load, such as a point load, uniform load, or linearly increasing load (triangular distribution). For loads where the length of the footing is greater than 5 times the width, such as for strip footings, the stress distribution is considered to be plane strain. This means that the horizontal strain of the elastic soil occurs only in the direction perpendicular to the long axis of the footing.

Although equations and charts based on the theory of elasticity are often used to determine the change in soil stress, soil is not an elastic material. For example, if a heavy foundation load is applied to a soil deposit, there will be vertical deformation of the soil in response to this load. If this heavy load is removed, the soil will rebound but not return to its original height because soil is not elastic. However, it has been stated that as long as the factor of safety against shear failure exceeds about 3, then stresses imposed by the foundation load are roughly equal to the values computed from elastic theory.

In 1885, Boussinesq published equations based on the theory of elasticity. For a surface point load (Q) applied at the ground surface such as shown in Figure 6.14, the vertical stress increase at any depth (z) and distance (r) from the point load can be calculated by using the following Boussinesq (1885) equation:

$$\sigma_z = \Delta\sigma_v = \frac{3Qz^3}{2\pi(r^2 + z^2)^{5/2}} \tag{6.17}$$

If there is a uniform line load Q (force per unit length), the vertical stress increase at a depth z and distance r from the line load would be:

$$\sigma_z = \Delta\sigma_v = \frac{2Qz^3}{\pi(r^2 + z^2)^2} \tag{6.18}$$

In 1935, Newmark performed an integration of Eq. (6.18) and derived an equation to determine the vertical stress increase (σ_z) under the corner of a loaded area. Convenient charts have been developed based on the Newmark (1935) equation. For example, Figure 6.15 shows the "pressure bulbs" (also known as isobars) beneath a square footing and a strip footing. To determine the change in vertical stress (i.e., σ_z or $\Delta\sigma_v$) at any point below the footing, multiply the value from Figure 6.15 times q_o, where q_o = uniform applied footing pressure.

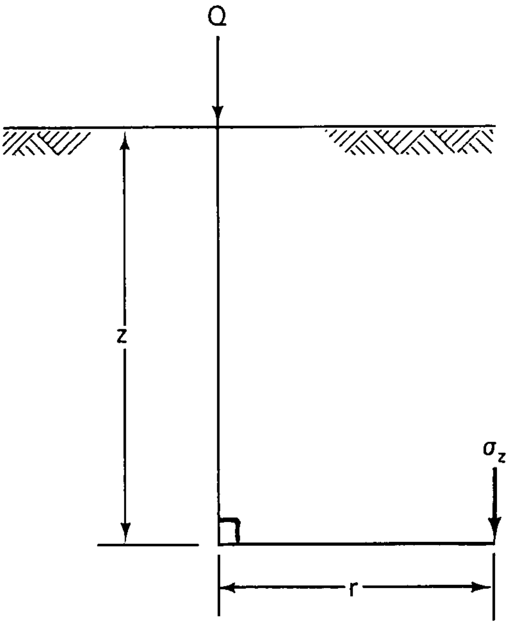


FIGURE 6.14 Definition of terms for Eqs. (6.17) and (6.18).

Figure 6.16 can be easily used to determine the pressure at the edge of a footing or can be used to determine the pressure under the center of the footing as described below. The values of m and n must be calculated. The value m is defined as the width of the loaded area (x) divided by the depth to where the vertical stress increase (σ_z) is to be calculated. The value n is defined as the length of the loaded area (y) divided by the depth (z). The chart is entered with the value of n and upon intersecting the desired m curve, the influence value (I) is then obtained from the vertical axis. As indicated in Fig. 6.16, vertical stress increase (σ_z) is then calculated as the loaded area pressure (q_o) times the influence value (I). Figure 6.16 can also be used to determine the vertical stress increase (σ_z) below the center of a rectangular loaded area. In this case, the rectangular loaded area would be divided into four parts and then Fig. 6.16 would be used to find the stress increase below the corner of one of the parts. By multiplying this stress by 4 (i.e., 4 parts), the vertical stress increase (σ_z) below the center of the total loaded area is obtained. This type of analysis is possible because of the principle of superposition for elastic materials. To find the vertical stress increase (σ_z) outside the loaded area, additional rectangular areas can be added and subtracted as needed in order to model the loading condition.

Figure 6.17 presents a chart for determining the change in vertical stress beneath a uniformly loaded circular area. Figure 6.18 shows a Newmark (1942) chart, which can be used to determine the vertical stress increase (σ_z) beneath a uniformly loaded area of any shape. There are numerous influence charts, each having a different influence value. Note that the chart in Fig. 6.18 has an influence value (I) of 0.005. The first step is to draw the loaded area onto the chart, using a scale where AB

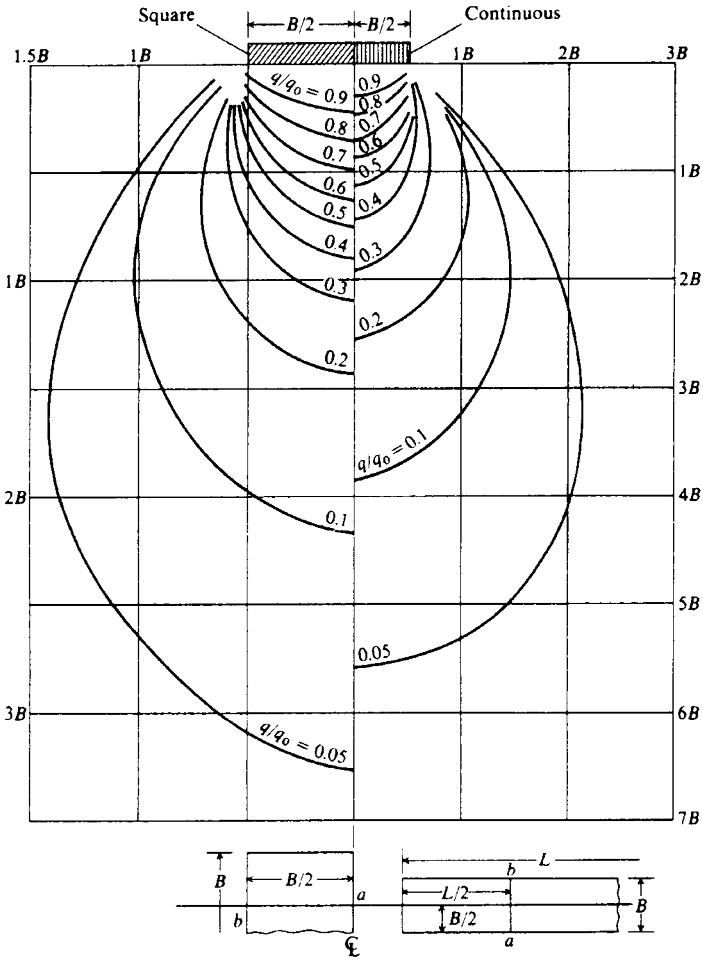


FIGURE 6.15 Pressure bulb beneath square footing and strip footing. The curves indicate the values of $\Delta\sigma_v/q_o$ beneath the footings, where q_o = uniform footing pressure (Note: applicable only along the line *ab* from center to edge of base). (*From J. E. Bowles, "Foundation Analysis and Design," 3d ed., McGraw-Hill Publishing Co., New York. Reproduced with permission of McGraw-Hill, Inc.*)

equals the depth z. The center of the chart must correspond to the point where the increase in vertical stress (σ_z) is desired. The increase in vertical stress (σ_z) is then calculated as: $\sigma_z = q_oIN$ where q_o = applied stress from the irregular area, I = influence value (0.005 for Fig. 6.18), and N = number of blocks within the irregular shaped area plotted on Fig. 6.18. When the value of N is being obtained, portions of blocks are also counted. Note that the entire procedure must be repeated if the increase in vertical stress (σ_z) is needed at a different depth.

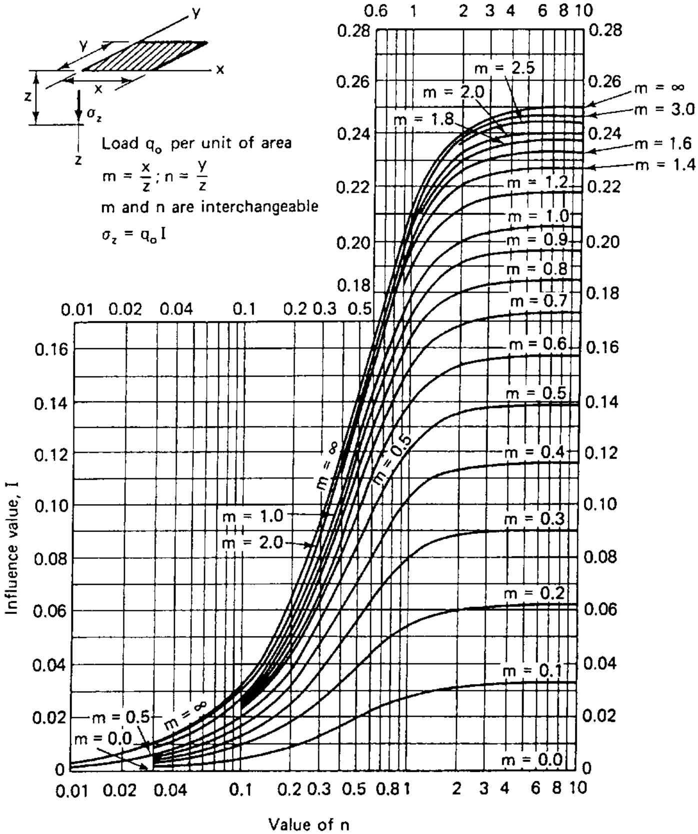


FIGURE 6.16 Chart for calculating the increase in vertical stress beneath the corner of a uniformly loaded rectangular area. (*From NAVFAC DM-7.1, 1982, Reproduced from R. D. Holtz and W. D. Kovacs, "An Introduction to Geotechnical Engineering," Prentice-Hall, Inc., Englewood Cliffs, NJ.*)

6.5 SETTLEMENT ANALYSES

Settlement can be defined as the permanent downward displacement of the foundation. There are two basic types of settlement, as follows:

6.5.1 Settlement Due Directly to the Weight of the Structure

The first type of settlement is directly caused by the weight of the structure. For example, the weight of a building may cause compression of an underlying sand deposit (Art. 6.5.7) or consolidation of an underlying clay layer (Art. 6.5.6). Often the settlement analysis is based on the actual dead load of the structure. The dead load is defined as the structural weight due to beams, columns, floors, roofs, and

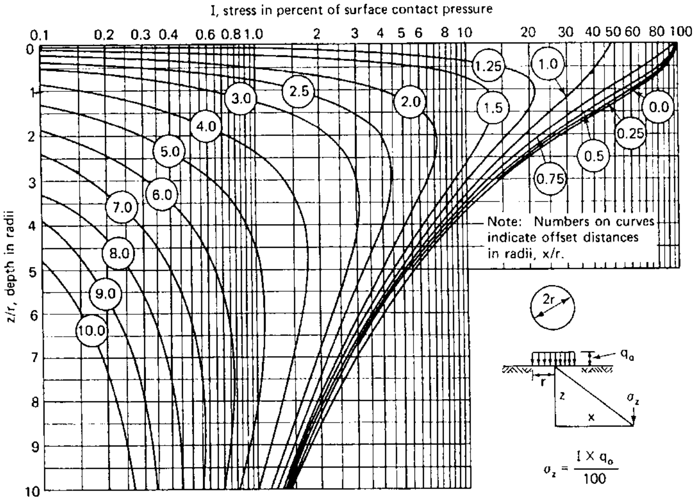


FIGURE 6.17 Chart for calculating the increase in vertical stress beneath a uniformly loaded circular area. (*From NAVFAC DM-7.1, 1982, Reproduced from R. D. Holtz and W. D. Kovacs, "An Introduction to Geotechnical Engineering," Prentice-Hall, Inc., Englewood Cliffs, NJ.*)

other fixed members. The dead load does not include nonstructural items. Live loads are defined as the weight of nonstructural members, such as furniture, occupants, inventory, and snow. Live loads can also result in settlement of the structure. For example, if the proposed structure is a library, then the actual weight of the books (a live load) should be included in the settlement analyses. Likewise, for a proposed warehouse, it may be appropriate to include the actual weight of anticipated stored items in the settlement analyses. In other projects where the live loads represent a significant part of the loading, such as large electrical transmission towers that will be subjected to wind loads, the live load (wind) may also be included in the settlement analysis. Considerable experience and judgment are required to determine the load that is to be used in the settlement analyses.

6.5.2 Settlement Due to Secondary Influences

The second basic type of settlement of a building is caused by secondary influence, which may develop at a time long after the completion of the structure. This type of settlement is not directly caused by the weight of the structure. For example, the foundation may settle as water infiltrates the ground and causes unstable soils to collapse (i.e., collapsible soil, Art. 6.5.5). The foundation may also settle due to yielding of adjacent excavations or the collapse of limestone cavities or underground mines and tunnels. Other causes of settlement that would be included in this category are natural disasters, such as settlement caused by earthquakes or undermining of the foundation from floods.

Subsidence is usually defined as a sinking down of a large area of the ground surface. Subsidence could be caused by the extraction of oil or groundwater that

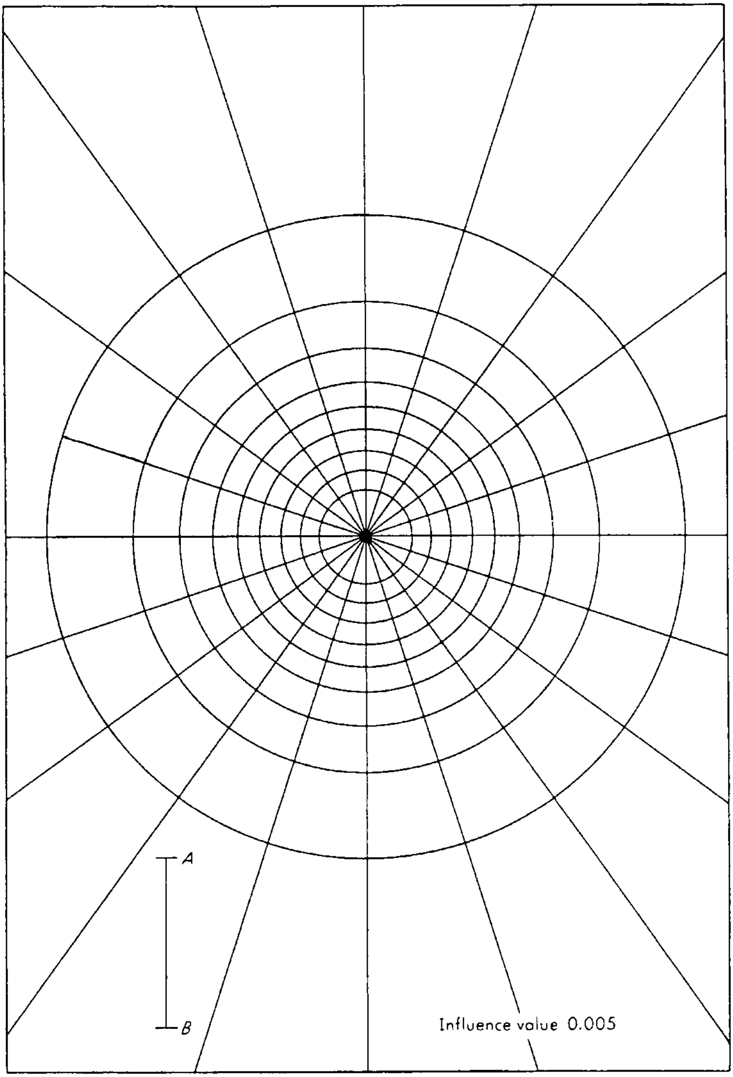


FIGURE 6.18 Newmark chart for calculating the increase in vertical stress beneath a uniformly loaded area of any shape. (*From N. M. Newmark, "Influence Charts for Computation of Stresses in Elastic Foundations," Univ. Illinois Expt. Sta. Bull. 338. Reproduced from J. E. Bowles, "Foundation Analysis and Design," 3d ed., McGraw-Hill Publishing Co., New York.*)

leads to a compression of the underlying porous soil or rock structure. Since subsidence is due to a secondary influence (extraction of oil or groundwater), its effect on the structure would be included in the second basic type of settlement.

6.5.3 Foundation Design Parameters

Determining the settlement of the structure is one of the primary obligations of the geotechnical engineer. In general, three parameters are required: maximum total settlement (ρ_{max}), maximum differential settlement (Δ), and rate of settlement. Another parameter that may be useful in the design of the foundation is the maximum angular distortion (δ/L), defined as the differential settlement between two points divided by the distance between them. Figure 6.19 illustrates the maximum total settlement (ρ_{max}), maximum differential settlement (Δ), and maximum angular distortion (δ/L) of a foundation. Note in Fig. 6.19 that the maximum angular distortion (δ/L) does not necessarily occur at the location of maximum differential settlement (Δ).

6.5.4 Allowable Settlement

The allowable settlement is defined as the acceptable amount of settlement of the structure and it usually includes a factor of safety. The allowable settlement depends on many factors, including the following (D. P. Coduto, "Foundation Design, Principles and Practices," Prentice-Hall, Inc., Englewood Cliffs, N.J.):

The Type of Construction. For example, wood-frame buildings with wood siding would be much more tolerant than unreinforced brick buildings.

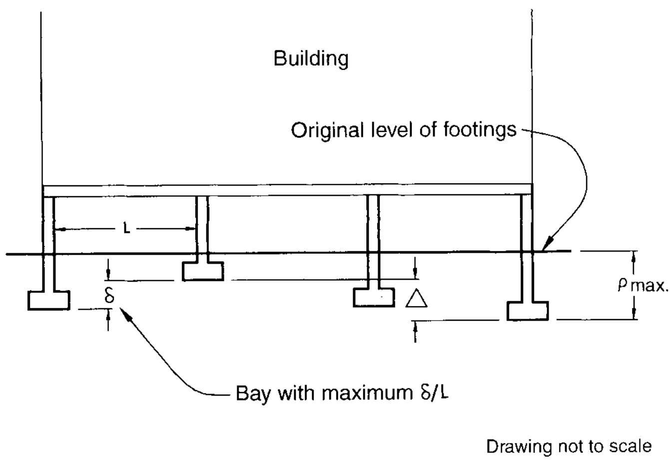


FIGURE 6.19 Diagram illustrating the definitions of maximum total settlement (ρ_{max}), maximum differential settlement (Δ), and maximum angular distortion (δ/L).

The Use of the Structure. Even small cracks in a house might be considered unacceptable, whereas much larger cracks in an industrial building might not even be noticed.

The Presence of Sensitive Finishes. Tile or other sensitive finishes are much less tolerant of movements.

The Rigidity of the Structure. If a footing beneath part of a very rigid structure settles more than the others, the structure will transfer some of the load away from the footing. However, footings beneath flexible structures must settle much more before any significant load transfer occurs. Therefore, a rigid structure will have less differential settlement than a flexible one.

Aesthetic and Serviceability Requirements. The allowable settlement for most structures, especially buildings, will be governed by aesthetic and serviceability requirements, not structural requirements. Unsightly cracks, jamming doors and windows, and other similar problems will develop long before the integrity of the structure is in danger.

Another example of allowable settlements for buildings is Table 6.13, where the allowable foundation displacement has been divided into three categories: total settlement, tilting, and differential movement. Table 6.13 indicates that those structures that are more flexible (such as simple steel frame buildings) or have more rigid

TABLE 6.13 Allowable Settlement

Type of movement	Limiting factor	Maximum settlement
Total settlement	Drainage	15–30 cm (6–12 in)
	Access	30–60 cm (12–24 in)
	Probability of nonuniform settlement:	
	Masonry walled structure	2.5–5 cm (1–2 in)
	Framed structures	5–10 cm (2–4 in)
	Smokestacks, silos, mats	8–30 cm (3–12 in)
Tilting	Stability against overturning	Depends on H and W
	Tilting of smokestacks, towers	0.004 L
	Rolling of trucks, etc.	0.01 L
	Stacking of goods	0.01 L
	Machine operation—cotton loom	0.003 L
	Machine operation—turbogenerator	0.0002 L
	Crane rails	0.003 L
	Drainage of floors	0.01–0.02 L
Differential movement	High continuous brick walls	0.0005–0.001 L
	One-story brick mill building, wall cracking	0.001–0.002 L
	Plaster cracking (gypsum)	0.001 L
	Reinforced concrete building frame	0.0025–0.004 L
	Reinforced concrete building curtain walls	0.003 L
	Steel frame, continuous	0.002 L
	Simple steel frame	0.005 L

L = distance between adjacent columns that settle different amounts, or between any two points that settle differently. Higher values are for regular settlements and more tolerant structures. Lower values are for irregular settlement and critical structures. H = height and W = width of structure.

Source: G. F. Sowers, ''Shallow Foundations,'' ch. 6 of ''Foundation Engineerings,'' ed. G. A. Leonards, McGraw-Hill Publishing Co., New York.

foundations (such as mat foundations) can sustain larger values of total settlement and differential movement.

6.5.5 Collapsible Soil

Collapsible soil can be defined as soil that is susceptible to a large and sudden reduction in volume upon wetting. Collapsible soil usually has a low dry density and low moisture content. Such soil can withstand a large applied vertical stress with a small compression, but then experience much larger settlements after wetting, with no increase in vertical pressure. Collapsible soil falls within the second basic category of settlement, which is settlement of the structure due to secondary influences.

In the southwestern United States, collapsible soil is probably the most common cause of settlement. The category of collapsible soil would include the settlement of debris, uncontrolled fill, deep fill, or natural soil, such as alluvium or colluvium. In general, there has been an increase in damage due to collapsible soil, probably because of the lack of available land in many urban areas. This causes development of marginal land, which may contain deposits of dumped fill or deposits of natural collapsible soil. Also, substantial grading can be required to develop level building pads, which results in more areas having deep fill.

The oedometer (also known as a consolidometer) is the primary laboratory equipment used to study the settlement behavior of soil. The oedometer test should only be performed on undisturbed soil specimens, or in the case of studies of fill behavior, on specimens compacted to anticipated field and moisture conditions. Figures 6.20 and 6.21 present the results of a collapse test performed on a soil

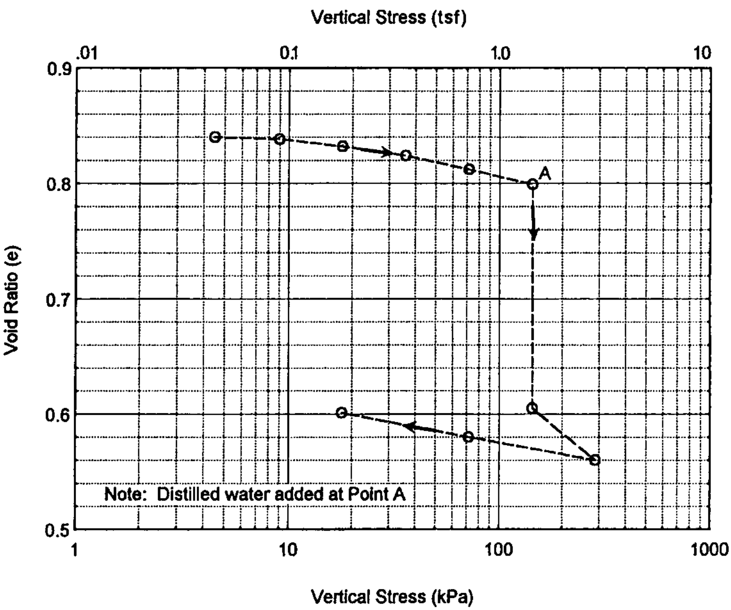


FIGURE 6.20 Laboratory collapse test results for a silty sand. The soil was loaded in the oedometer to a pressure of 144 kPa and then inundated with distilled water.

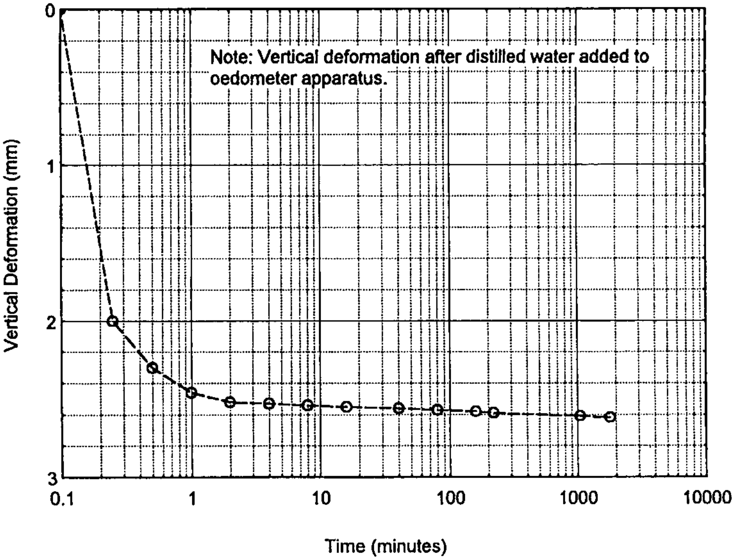


FIGURE 6.21 Laboratory collapse test results for a silty sand. This plot shows the vertical deformation versus time after the soil specimen was inundated with distilled water.

specimen by using the laboratory oedometer equipment. The soil specimen was loaded in the oedometer to a vertical pressure of 144 kPa (3000 psf) and then inundated with distilled water. Figure 6.20 shows the load-settlement behavior of the soil specimen, and Fig. 6.21 shows the amount of vertical deformation (collapse) as a function of time after inundation. The percent collapse is defined as the change in height of the soil specimen after inundation divided by the initial height of the soil specimen.

There are many different methods for dealing with collapsible soil. If there is a shallow deposit of natural collapsible soil, the deposit can be removed and recompacted during the grading of the site. In some cases, the soil can be densified (such as by compaction grouting) to reduce the collapse potential of the soil. Another method for dealing with collapsible soil is to flood the building footprint or force water into the collapsible soil stratum by using wells. As the wetting front moves through the ground, the collapsible soil will densify and reach an equilibrium state. Flooding or forcing water into collapsible soil should not be performed if there are adjacent buildings, due to the possibility of damaging these structures. Also, after the completion of the flooding process, subsurface exploration and laboratory testing should be performed to evaluate the effectiveness of the process.

There are also foundation options that can be used for sites containing collapsible soil. A deep foundation system, which derives support from strata below the collapsible soil, could be constructed. Also, post-tensioned foundations or mat slabs can be designed and installed to resist the larger anticipated settlement from the collapsible soil.

6.5.6 Settlement of Cohesive and Organic Soils

Cohesive and organic soil can be susceptible to a large amount of settlement from structural loads. It is usually the direct weight of the structure that causes settlement of the cohesive or organic soil. The settlement of saturated clay or organic soil can have three different components: immediate (also known as "initial settlement"), primary consolidation, and secondary compression.

Immediate Settlement. In most situations, surface loading causes both vertical and horizontal strains. This is referred to as two- or three-dimensional loading. Immediate settlement is due to undrained shear deformations, or in some cases contained plastic flow, caused by the two- or three-dimensional loading. Common examples of three-dimensional loading are from square footings and round storage tanks. Many different methods are available to determine the amount of immediate settlement for two- or three-dimensional loadings. Examples include field plate load tests, equations based on the theory of elasticity, and the stress path method (see R. W. Day, "Geotechnical and Foundation Engineering: Design and Construction," McGraw-Hill Publishing Co., New York).

Primary Consolidation. The increase in vertical pressure due to the weight of the structure constructed on top of saturated soft clays and organic soil will initially be carried by the pore water in the soil. This increase in pore water pressure is known as an excess pore water pressure (u_e). The excess pore water pressure will decrease with time as water slowly flows out of the cohesive soil. This flow of water from cohesive soil (which has a low permeability) as the excess pore water pressures slowly dissipate is known as **primary consolidation,** or simply **consolidation.** As the water slowly flows from the cohesive soil, the structure settles as the load is transferred to the soil particle skeleton, thereby increasing the effective stress of the soil. Consolidation is a time-dependent process that may take many years to complete.

Based on the stress history of saturated cohesive soils, they are considered to be either underconsolidated, normally consolidated, or overconsolidated. The overconsolidation ratio (OCR) is used to describe the stress history of cohesive soil, and it is defined as: OCR $= \sigma'_{vm}/\sigma'_{vo}$, where: σ'_{vm} or σ'_p = maximum past pressure (σ'_{vm}), also known as the preconsolidation pressure (σ'_p), which is equal to the highest previous vertical effective stress that the cohesive soil was subjected to and consolidated under, and σ'_{vo} or σ'_v = existing vertical effective stress. In terms of the stress history of a cohesive soil, there are three possible conditions:

1. **Underconsolidated** (OCR < 1). A saturated cohesive soil is considered underconsolidated if the soil is not fully consolidated under the existing overburden pressure and excess pore water pressures (u_e) exist within the soil. Underconsolidation occurs in areas where a cohesive soil is being deposited very rapidly and not enough time has elapsed for the soil to consolidate under its own weight.
2. **Normally Consolidated** (OCR $= 1$). A saturated cohesive soil is considered normally consolidated if it has never been subjected to an effective stress greater than the existing overburden pressure and if the deposit is completely consolidated under the existing overburden pressure.
3. **Overconsolidated or Preconsolidated** (OCR > 1): A saturated cohesive soil is considered overconsolidated if it has been subjected in the past to a vertical

effective stress greater than the existing vertical effective stress. An example of a situation that creates an overconsolidated soil is where a thick overburden layer of soil has been removed by erosion over time. Other mechanisms, such as changes in groundwater elevation and changes in soil structure, can create an overconsolidated soil.

For structures constructed on top of saturated cohesive soil, determining the overconsolidation ratio of the soil is very important in the settlement analysis. For example, if the cohesive soil is underconsolidated, then considerable settlement due to continued consolidation owing to the soil's own weight as well as the applied structural load would be expected. On the other hand, if the cohesive soil is highly overconsolidated, then a load can often be applied to the cohesive soil without significant settlement.

The oedometer apparatus is used to determine the consolidation properties of saturated cohesive soil. The typical testing procedure consists of placing an undisturbed specimen within the apparatus, applying a vertical seating pressure to the laterally confined specimen, and then submerging the specimen in distilled water. The specimen is then subjected to an incremental increase in vertical stress, with each pressure remaining on the specimen for a period of 24 hr. Plotting the vertical stress versus void ratio of the soil often yields a plot similar to Fig. 6.22. The plot

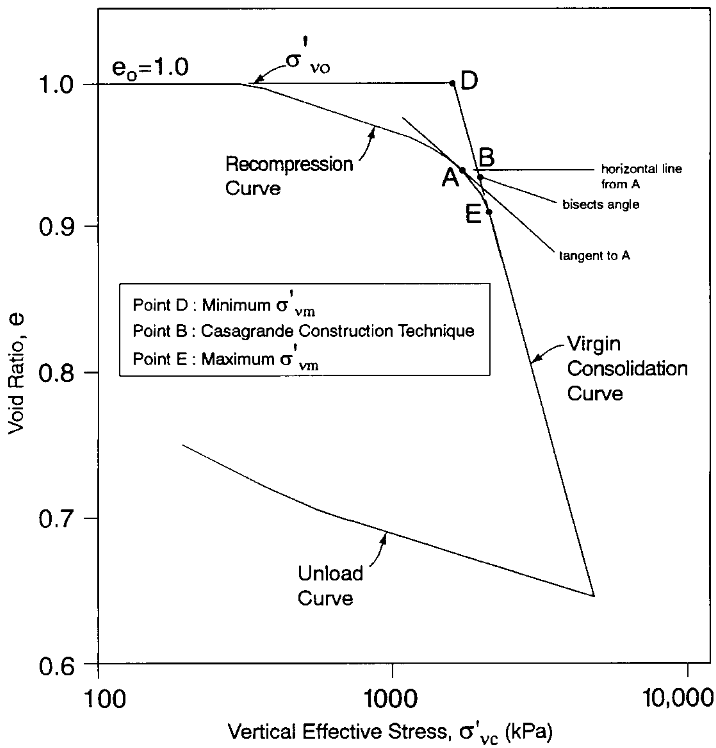


FIGURE 6.22 Example of a consolidation curve. The Casagrande construction technique for determining the maximum past pressure is also shown on this figure.

is known as the consolidation curve and consists of two important segments, the recompression curve (defined by the recompression index C_r) and the virgin consolidation curve (defined by the compression index C_c). Figure 6.22 also illustrates the Casagrande construction technique, which can be used to determine the maximum past pressure (σ'_{vm}).

Using the calculated values of C_r and C_c, the primary consolidation settlement (s_c) due to an increase in load ($\Delta\sigma_v$) can be determined from the following equations:

1. For underconsolidated soil (OCR $<$ 1):

$$s_c = C_c \frac{H_o}{1 + e_o} \log \frac{\sigma'_{vo} + \Delta\sigma'_v + \Delta\sigma_v}{\sigma'_{vo}} \tag{6.19}$$

2. For normally consolidated soil (OCR $=$ 1):

$$s_c = C_c \frac{H_o}{1 + e_o} \log \frac{\sigma'_{vo} + \Delta\sigma_v}{\sigma'_{vo}} \tag{6.20}$$

3. For overconsolidated soil (OCR $>$ 1):
Case I: $\sigma'_{vo} + \Delta\sigma_v \leq \sigma'_{vm}$

$$s_c = C_r \frac{H_o}{1 + e_o} \log \frac{\sigma'_{vo} + \Delta\sigma_v}{\sigma'_{vo}} \tag{6.21}$$

Case II: $\sigma'_{vo} + \Delta\sigma_v > \sigma'_{vm}$

$$s_c = C_r \frac{H_o}{1 + e_o} \log \frac{\sigma'_{vm}}{\sigma'_{vo}} + C_c \frac{H_o}{1 + e_o} \log \frac{\sigma'_{vo} + \Delta\sigma_v}{\sigma'_{vm}} \tag{6.22}$$

where s_c = settlement due to primary consolidation caused by an increase in load
C_c = compression index, obtained from the virgin consolidation curve (Fig. 6.22)
C_r = recompression index, obtained from the recompression portion of the consolidation curve (Fig. 6.22)
H_o = initial thickness of the in-situ saturated cohesive soil layer
e_o = initial void ratio of the in-situ saturated cohesive soil layer
σ'_{vo} = initial vertical effective stress of the in-situ soil (see Art. 6.4.1)
$\Delta\sigma'_v$ = for an underconsolidated soil (this represents the increase in vertical effective stress that will occur as the cohesive soil consolidates under its own weight)
σ'_{vm} = maximum past pressure, also known as the preconsolidation pressure (σ'_p), which is obtained from the consolidation curve using the Casagrande construction technique (see Fig. 6.22)
$\Delta\sigma_v$ = increase in load, typically due to the construction of a building or the construction of a fill layer at ground surface

The value of $\Delta\sigma_v$ (also known as σ_z) can be obtained from stress distribution theory as discussed in Art. 6.4.2. Note that a drop in the groundwater table or a reduction in pore water pressure can also result in an increase in load on the cohesive soil.

For overconsolidated soil, there are two possible cases that can be used to calculate the amount of settlement. The first case occurs when the existing vertical effective stress (σ'_{vo}) plus the increase in vertical stress ($\Delta\sigma_v$) due to the proposed building weight does not exceed the maximum past pressure (σ'_{vm}). For this first case, there will only be recompression of the cohesive soil.

For the second case, the sum of the existing vertical effective stress (σ'_{vo}) plus the increase in vertical stress ($\Delta\sigma_v$) due to the proposed building weight exceeds the maximum past pressure (σ'_{vm}). For the second case, there will be virgin consolidation of the cohesive soil. Given the same cohesive soil and identical field conditions, the settlement due to the second case will be significantly more than the first case.

As previously mentioned, primary consolidation is a time-dependent process that can take many years to complete. The rate of consolidation can be estimated using the Terzaghi theory of consolidation (see K. Terzaghi and R. B. Peck, "Soil Mechanics in Engineering Practice," John Wiley & Sons, Inc., New York):

Secondary Compression. The final component of settlement is due to secondary compression, which is that part of the settlement that occurs after essentially all of the excess pore water pressures have dissipated (i.e., settlement that occurs at constant effective stress). The usual assumption is that secondary compression does not start until after primary consolidation is complete. The amount of secondary compression is often neglected because it is rather small compared to the primary consolidation settlement. However, secondary compression can constitute a major part of the total settlement for peat or other highly organic soil (see R. D. Holtz and W. D. Kovacs, "An Introduction to Geotechnical Engineering," Prentice-Hall, Inc., Englewood Cliffs, NJ).

The final calculation for estimating the maximum settlement ($\rho_{\max}$) of the insitu cohesive soil would be to add together the three components of settlement, or:

$$\rho_{\max} = s_i + s_c + s_s \tag{6.23}$$

where $\rho_{\max}$ = maximum settlement over the life of the structure
s_i = immediate settlement
s_c = primary consolidation settlement
s_s = secondary compression settlement

6.5.7 Settlement of Granular Soil

A major difference between saturated cohesive soil and granular soil is that the settlement of cohesionless soil is not time dependent. Because of the generally high permeability of granular soil, the settlement usually occurs as the load is applied during the construction of the building. Many different methods can be used to determine the settlement of granular soil, such as plate load tests, laboratory testing of undisturbed soil samples, equations based on the theory of elasticity, and empirical correlations. For example, Fig. 6.23 shows a chart that presents an empirical correlation between the measured *N* value (obtained from the Standard Penetration Test, see Art. 6.2.4) and the allowable soil pressure (tsf) that will produce a settlement of the footing of 1 in (2.5 cm).

As an example of the use of Fig. 6.23, suppose a site contains a sand deposit and the proposed structure can be subjected to a maximum settlement ($\rho_{\max}$) of 1.0 in (2.5 cm). If the measured *N* value from the Standard Penetration Test = 10 and the width of the proposed footings = 5 ft (1.5 m), then the allowable soil pressure = 1 tsf (100 kPa).

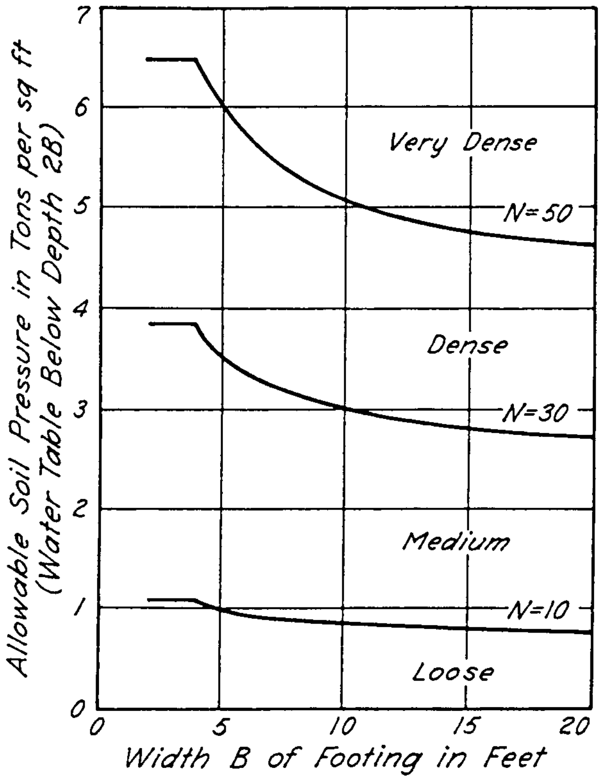


FIGURE 6.23 Allowable soil bearing pressures for footings on sand based on the Standard Penetration Test. (*From K. Terzaghi and R. B. Peck, "Soil Mechanics in Engineering Practice," 2d ed., John Wiley & Sons, Inc., New York. Reprinted with permission of John Wiley & Sons, Inc.*)

For measured *N* values other than those for which the curves are drawn in Fig. 6.23, the allowable soil pressure can be obtained by linear interpolation between curves. According to Terzaghi and Peck, if all of the footings are proportioned in accordance with the allowable soil pressure corresponding to Fig. 6.23, then the maximum settlement (ρ_{max}) of the foundation should not exceed 1 in (2.5 cm) and the maximum differential settlement (Δ) should not exceed 0.75 in (2 cm). Figure 6.23 was developed for the groundwater table located at a depth equal to or greater than a depth of 2*B* below the bottom of the footing. For conditions of a high groundwater table close to the bottom of the shallow foundation, the values obtained from Fig. 6.23 should be reduced by 50%.

6.6 BEARING CAPACITY ANALYSES

A bearing capacity failure is defined as a foundation failure that occurs when the shear stresses in the soil exceed the shear strength of the soil. Bearing capacity failures of foundations can be grouped into three categories (A. B. Vesić, "Bearing Capacity of Deep Foundations in Sand," *Highway Research Record,* no. 39):

1. **General Shear** (Fig. 6.24). As shown in Fig. 6.24, a general shear failure involves total rupture of the underlying soil. There is a continuous shear failure of the soil (solid lines) from below the footing to the ground surface. When the load is plotted versus settlement of the footing, there is a distinct load at which the foundation fails (solid circle), and this is designated Q_{ult}. The value of Q_{ult} divided by the width (B) and length (L) of the footing is considered to be the "ultimate bearing capacity" (q_{ult}) of the footing. The ultimate bearing capacity has been defined as the bearing stress that causes a sudden catastrophic failure of the foundation. Note in Fig. 6.24 that a general shear failure ruptures and pushes up the soil on both sides of the footing. For actual failures it the field, the soil is often pushed up on only one side of the footing with subsequent tilting of the structure. A general shear failure occurs for soils that are in a dense or hard state.
2. **Punching Shear** (Fig. 6.25). As shown in Fig. 6.25, a punching shear failure does not develop the distinct shear surfaces associated with a general shear failure. For punching shear, the soil outside the loaded area remains relatively uninvolved and there is minimal movement of soil on both sides of the footing.

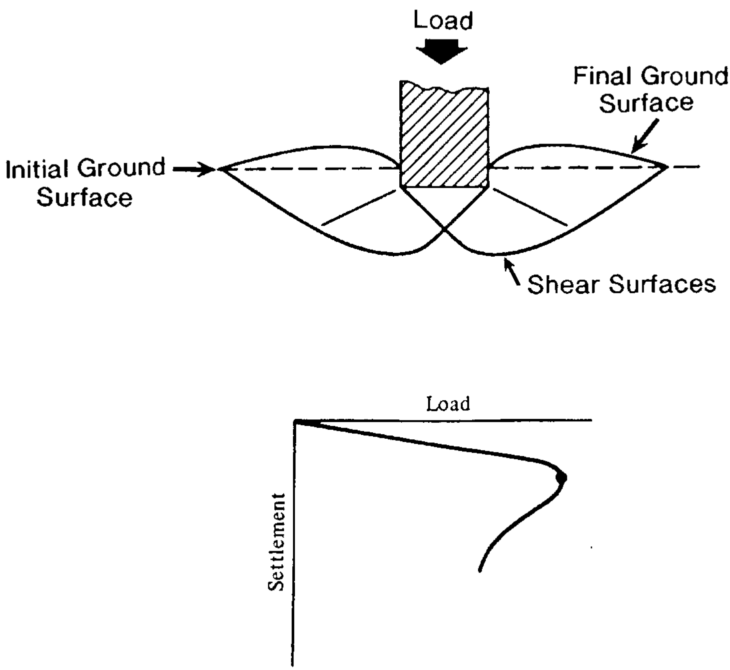


FIGURE 6.24 General shear foundation failure for soil in a dense or hard state. (*Adapted from A. B. Vesić, "Bearing Capacity of Deep Foundations in Sand," Highway Research Record, no. 39.*)

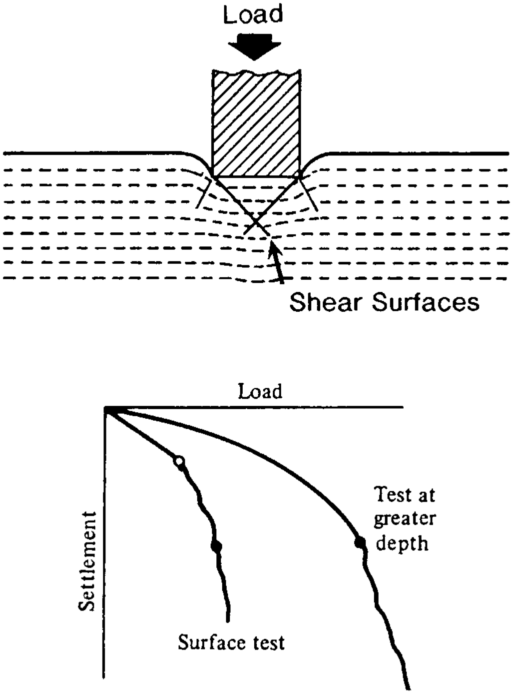


FIGURE 6.25 Punching shear foundation failure for soil in a loose or soft state. (*Adapted from A. B. Vesić, "Bearing Capacity of Deep Foundations in Sand," Highway Research Record, no. 39.*)

The process of deformation of the footing involves compression of soil directly below the footing as well as the vertical shearing of soil around the footing perimeter. As shown in Fig. 6.25, the load settlement curve does not have a dramatic break, and for punching shear, the bearing capacity is often defined as the first major nonlinearity in the load-settlement curve (open circle). A punching shear failure occurs for soils that are in a loose or soft state.

3. **Local Shear Failure** (Fig. 6.26). As shown in Fig. 6.26, local shear failure involves rupture of the soil only immediately below the footing. There is soil bulging on both sides of the footing, but the bulging is not as significant as in general shear. Local shear failure can be considered as a transitional phase between general shear and punching shear. Because of the transitional nature of local shear failure, the bearing capacity could be defined as the first major nonlinearity in the load-settlement curve (open circle) or at the point where the settlement rapidly increases (solid circle). A local shear failure occurs for soils that have a medium density or firm state.

The documented cases of bearing capacity failures indicate that usually the following three factors (separately or in combination) are the cause of the failure:

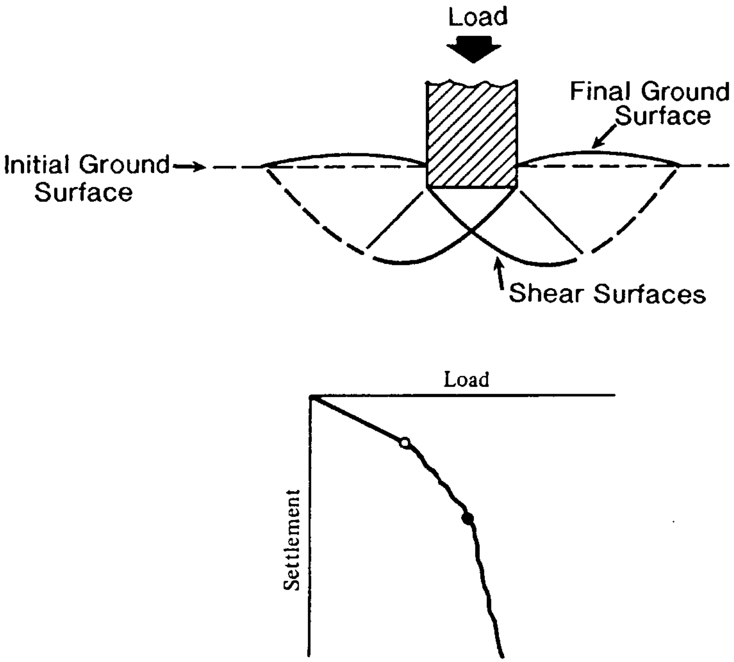


FIGURE 6.26 Local shear foundation failure, which is a transitional phase between general shear and punching shear failures. (*Adapted from A. B. Vesić, "Bearing Capacity of Deep Foundations in Sand," Highway Research Record, no. 39.*)

1. There was an overestimation of the shear strength of the underlying soil.
2. The actual structural load at the time of the bearing capacity failure was greater than that assumed during the design phase.
3. The site was altered, such as the construction of an adjacent excavation, which resulted in a reduction in support and a bearing capacity failure.

A famous case of a bearing capacity failure is the Transcona grain elevator, located at Transcona, Manitoba, Canada, near Winnipeg. Figure 6.27 shows the October 1913 failure of the grain elevator. At the time of failure, the grain elevator was essentially fully loaded. The foundation had been constructed on clay that was described as a stiff clay. Note in Fig. 6.27 that the soil has been pushed up on only one side of the foundation, with subsequent tilting of the structure.

6.6.1 Bearing Capacity for Shallow Foundations

As indicated in Table 6.2, common types of shallow foundations include spread footings for isolated columns, combined footings for supporting the load from more than one structural unit, strip footings for walls, and mats or raft foundations constructed at or near ground surface. Shallow footings often have an embedment that is less than the footing width.

FIGURE 6.27 Transcona grain elevator bearing capacity failure.

Bearing Capacity Equation. The most commonly used bearing capacity equation is the equation developed by Terzaghi (''Theoretical Soil Mechanics,'' John Wiley & Sons, Inc., New York). For a uniform vertical loading of a strip footing, Terzaghi assumed a general shear failure (Fig. 6.24) in order to develop the following bearing capacity equation:

$$q_{\text{ult}} = \frac{Q_{\text{ult}}}{BL} = cN_c + \tfrac{1}{2}\,\gamma_t B N_\gamma + \gamma_t D_f N_q \tag{6.24}$$

where q_{ult} = ultimate bearing capacity for a strip footing
Q_{ult} = vertical load causing a general shear failure of the underlying soil (Fig. 6.24)
B = width of the strip footing
L = length of the strip footing
γ_t = total unit weight of the soil
D_f = vertical distance from the ground surface to the bottom of the strip footing
c = cohesion of the soil underlying the strip footing
N_c, N_γ, and N_q = dimensionless bearing capacity factors

In order to calculate the allowable bearing pressure (q_{all}), the following equation is used: $q_{\text{all}} = q_{\text{ult}}/F$, where q_{all} = allowable bearing pressure, q_{ult} = ultimate bearing capacity from Eq. (6.24), and F = factor of safety (typically $F = 3$). This allowable bearing pressure often has to be reduced in order to prevent excessive settlement of the foundation. In addition, building codes often list allowable bearing pressures versus soil or rock types, such as Table 6.14, which presents the allowable bearing pressures (q_{all}) from the ''Uniform Building Code'' (1997).

TABLE 6.14 Allowable Bearing Pressures

Material type	Allowable bearing pressure[a]	Maximum allowable bearing pressure[b]
Massive crystalline bedrock	4,000 psf (200 kPa)	12,000 psf (600 kPa)
Sedimentary and foliated rock	2,000 psf (100 kPa)	6,000 psf (300 kPa)
Gravel and sandy gravel (GW, GP)	2,000 psf (100 kPa)	6,000 psf (300 kPa)
Nonplastic soil: sands, silts, and NP silt (GM, SW, SP, SM)[c]	1,500 psf (75 kPa)	4,500 psf (220 kPa)
Plastic soil: silts and clays (ML, MH, SC, CL, CH)[c]	1,000 psf (50 kPa)	3,000 psf (150 kPa)[d]

[a] Minimum footing width and embedment depth equals 1 ft (0.3 m).
[b] An increase of 20% of the allowable bearing pressure is allowed for each additional foot (0.3 m) of width or depth up to the maximum allowable bearing pressures listed in Column 3. An exception is plastic soil, see note *d*.
[c] Group symbols from Table 6.8.
[d] No increase in the allowable bearing pressure is allowed for an increase in width of the footing.
For dense or stiff soils, allowable bearing values in this table are generally conservative. For very loose or very soft soils, the allowable bearing values may be too high.
Source: Data from ''Uniform Building Code'' (1997)

There are many charts, graphs, and figures that present bearing capacity factors developed by different engineers and researchers based on varying assumptions. For example, Fig. 6.28 presents bearing capacity factors N_c, N_γ, and N_q, which automatically incorporate allowance for punching and local shear failure. Another example is Fig. 6.29, which presents bearing capacity factors that have not been adjusted for punching or local shear failure. Figure 6.29 also presents the bearing capacity equations for square, rectangular, and circular footings. The equations for granular soil (i.e., cohesionless soil, $c = 0$) and for a total stress analysis for cohesive soil (i.e., $\phi = 0$ and $c = s_u$) are also shown in Fig. 6.29.

Other Footing Loads. In addition to the vertical load acting on the footing, it may also be subjected to a lateral load. A common procedure is to treat lateral loads separately and resist the lateral loads by using the soil pressure acting on the sides of the footing (passive pressure) and the frictional resistance along the bottom of the footing.

It is always desirable to design and construct shallow footings so that the vertical load is applied at the center of gravity of the footing. For combined footings that carry more than one vertical load, the combined footing should be designed and constructed so that the vertical loads are symmetric. There may be design situations where the footing is subjected to a moment, such as where there is a fixed-end connection between the building frame and the footing. This moment can be represented by a load Q that is offset a certain distance (known as the eccentricity) from the center of gravity of the footing. For other projects, there may be property line constraints and the load must be offset a certain distance (eccentricity) from the center of gravity of the footing. Because an eccentrically loaded footing will create a higher bearing pressure under one side as compared to the opposite side, one approach is to evaluate the actual pressure distribution beneath the footing. The usual procedure is to assume a rigid footing (hence linear pressure distribution) and use the section modulus ($\frac{1}{6}B^2$) in order to calculate the largest and lowest bearing pressure. For a footing having a width B, the largest (q') and lowest (q'') bearing pressures are as follows:

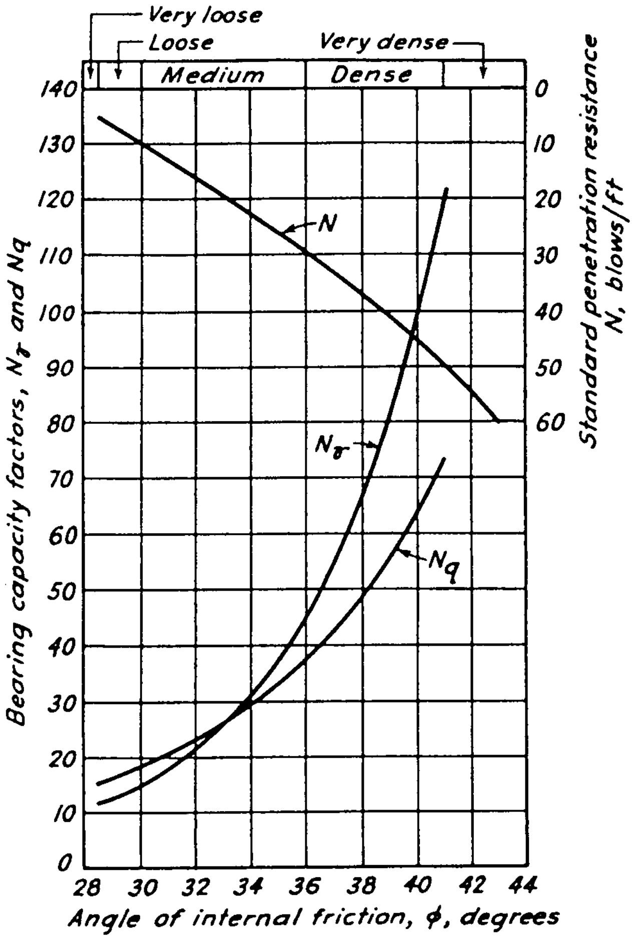


FIGURE 6.28 Bearing capacity factors N_γ and N_q, which automatically incorporate allowance for punching and local shear failure. (*Reproduced from R. B. Peck, W. E. Hanson, and T. H. Thornburn, "Foundation Engineering," John Wiley & Sons, Inc., New York, reproduced with permission of John Wiley & Sons, Inc.*)

$$q' = \frac{Q(B + 6e)}{B^2} \qquad q'' = \frac{Q(B - 6e)}{B^2} \tag{6.25}$$

where q' = largest bearing pressure underneath the footing, which is located along the same side of the footing as the eccentricity

q'' = lowest bearing pressure underneath the footing, which is located at the opposite side of the footing

Q = load applied to the footing (kN per linear m of footing length or lb per linear ft of footing length)

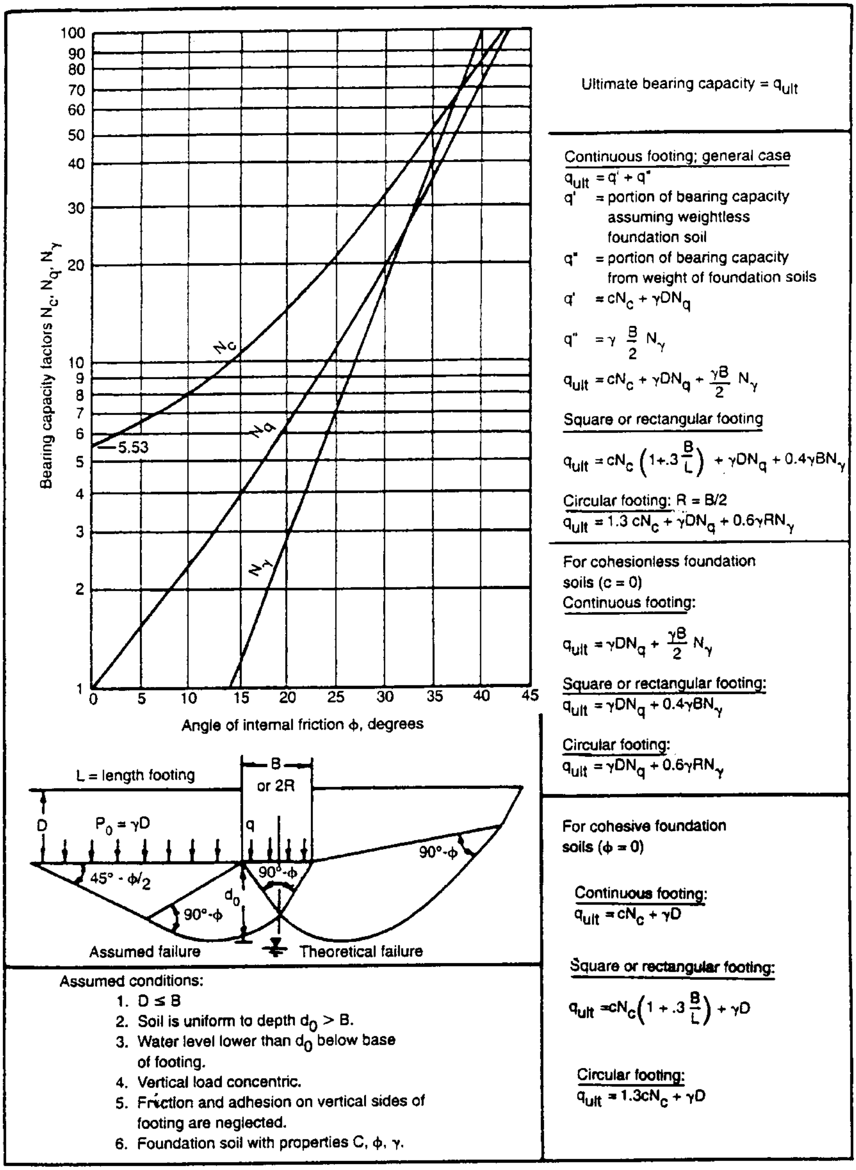


FIGURE 6.29 Bearing capacity factors N_γ, N_q, and N_c, which do not include allowance for punching or local shear failure. (Note: for local or punching shear of loose sands or soft clays, the value of ϕ to be used in this figure = $\tan^{-1}$ (0.67 tan ϕ) and the cohesion used in the bearing capacity equation = 0.67 c). (*Reproduced from NAVFAC DM-7.2, 1982.*)

e = eccentricity of the load Q; i.e., the lateral distance from Q to the center of gravity of the footing
B = width of the footing

A usual requirement is that the load (Q) must be located within the middle ⅓ of the footing. The above equations are only valid for this condition. The value of q' must not exceed the allowable bearing pressure (q_{all}).

For dense or stiff soils, allowable bearing values in Table 6.14 are generally conservative. For very loose or very soft soils, the allowable bearing values in Table 6.14 may be too high.

6.6.2 Bearing Capacity for Deep Foundations in Granular Soil

Deep foundations are used when the upper soil stratum is too soft, weak, or compressible to support the foundation loads. Deep foundations are also used when there is a possibility of the undermining of the foundation. For example, bridge piers are often founded on deep foundations to prevent a loss of support due to flood conditions which could cause river bottom scour. The most common types of deep foundations are piles and piers that support individual footings or mat foundations (Table 6.2). Piles are defined as relatively long, slender, column-like members often made of steel, concrete, or wood that are either driven into place or cast-in-place in predrilled holes. Common types of piles are as follows:

Batter Pile. A pile driven in at an angle inclined to the vertical to provide high resistance to lateral loads.

End-Bearing Pile. A pile whose support capacity is derived principally from the resistance of the foundation material on which the pile tip rests. End-bearing piles are often used when a soft upper layer is underlain by a dense or hard stratum. If the upper soft layer should settle, the pile could be subjected to down-drag forces, and the pile must be designed to resist these soil-induced forces.

Friction Pile. A pile whose support capacity is derived principally from the resistance of the soil friction and/or adhesion mobilized along the side of the pile. Friction piles are often used in soft clays where the end-bearing resistance is small because of punching shear at the pile tip.

Combined End-Bearing and Friction Pile. A pile that derives its support capacity from combined end-bearing resistance developed at the pile tip and frictional and/or adhesion resistance on the pile perimeter.

A **pier** is defined as a deep foundation system, similar to a cast-in-place pile, that consists of a column-like reinforced concrete member. Piers are often of large enough diameter to enable down-hole inspection. Piers are also commonly referred to as drilled shafts, bored piles, or drilled caissons.

Many other methods are available for forming deep foundation elements. Examples include earth stabilization columns, such as (NAVFAC DM-7.2, 1982):

Mixed-in-Place Piles. A mixed-in-place soil-cement or soil-lime pile.

Vibro-Replacement Stone Columns. Vibroflotation or other method is used to make a cylindrical, vertical hole that is filled with compacted gravel or crushed rock.

Grouted Stone Columns. Similar to the above but includes filling voids with bentonite-cement or water-sand-bentonite cement mixtures.

Concrete Vibro Columns. Similar to stone columns, but concrete is used instead of gravel.

Several different items are used in the design and construction of piles, including:

Engineering Analysis. Based on the results of subsurface exploration and laboratory testing, the bearing capacity of the deep foundation can be calculated in a similar manner to the previous section on shallow foundations. This section will describe the engineering analyses for deep foundations in granular and cohesive soil.

Field Load Tests. Prior to the construction of the foundation, a pile or pier could be load tested in the field to determine its carrying capacity. Because of the uncertainties in the design of piles based on engineering analyses, pile load tests are common. The pile load test can often result in a more economical foundation than one based solely on engineering analyses.

Application of Pile Driving Resistance. Often the pile driving resistance (i.e., blows per ft) is recorded as the pile is driven into place. When the anticipated bearing layer is encountered, the driving resistance (blows per ft) should substantially increase.

Specifications and Experience. Other factors that should be considered in the deep foundation design include governing building code or agency requirements and local experience.

End Bearing Pile for Granular Soil. For an end bearing pile or pier, the bearing capacity equation can be used to determine the ultimate bearing capacity (q_{ult}). When we compare the second and third term in Eq. (6.24), the value of B (width of pile) is much less than the embedment depth (D_f) of the pile. Therefore, the second term in Eq. (6.24) can be neglected. Assuming granular soil (c = 0), Eq. (6.24) reduces to the following:

$$q_{\text{ult}} = \frac{Q_p}{\text{area}} = \gamma_t D_f N_q = \sigma'_v N_q \tag{6.26}$$

where q_{ult} = the ultimate bearing capacity of the end-bearing pile or pier
Q_p = point resistance force
area = pile tip area (B^2 in the case of a square pile and πR^2 in the case of a round pile)
σ'_v = vertical effective stress at the pile tip
N_q = dimensionless bearing capacity factor

For drilled piers or piles placed in predrilled holes, the value of N_q can be obtained from Fig. 6.28 or 6.29 based on the friction angle (ϕ) of the granular soil located at the pile tip. However, for driven piles, the values of N_q listed in Figs. 6.28 and 6.29 are generally too conservative. Figure 6.30 presents a chart that provides the bearing capacity factor N_q from several different sources. Note in Fig. 6.30 that at $\phi = 30°$, N_q varies from about 30 to 150, while at $\phi = 40°$, N_q varies from about 100 to 1000. This is a tremendous variation in N_q values and is related to the different approaches used by the various researchers, where in some cases

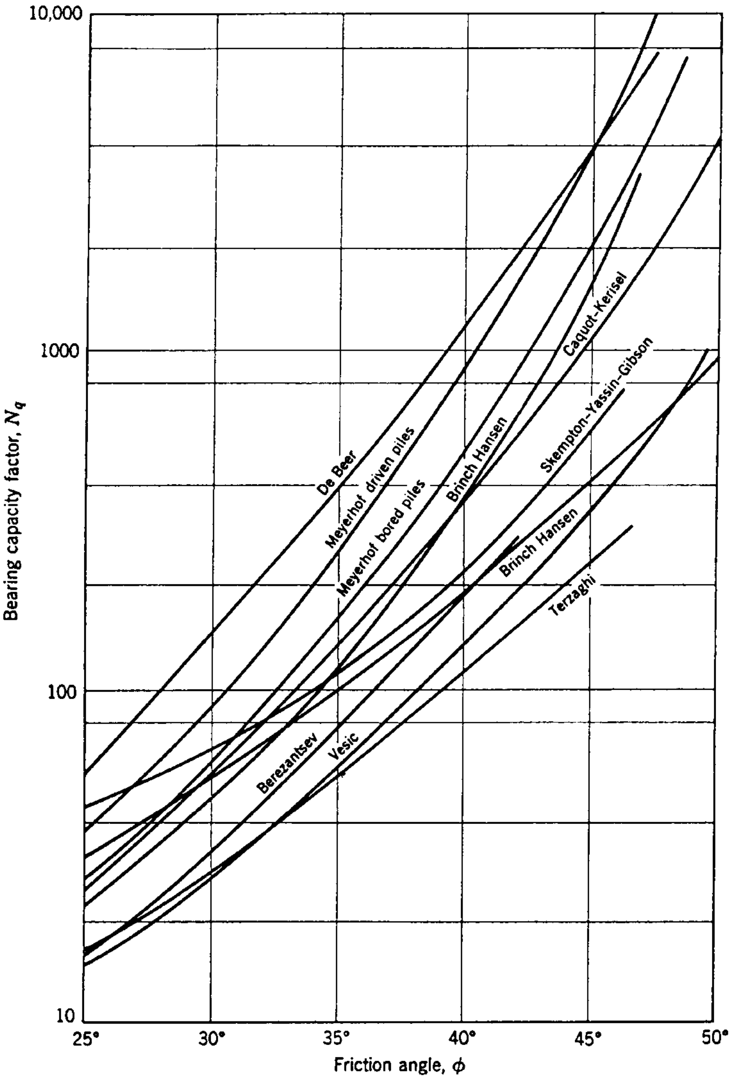


FIGURE 6.30 Bearing capacity factor N_q as recommended by various researchers for deep foundations. (*Originally from A. S. Vesić, "Ultimate Loads and Settlements of Deep Foundations in Sand," Duke University, Durham, NC.*)

the basis of the relationship shown in Fig. 6.30 is theoretical and in other cases the relationship is based on analysis of field data such as pile load tests. There is a general belief that the bearing capacity factor N_q is higher for driven piles than for shallow foundations. One reason for a higher N_q value is the effect of driving the pile, which displaces and densifies the cohesionless soil at the bottom of the pile. The densification could be due to both the physical process of displacing the soil and the driving vibrations. These actions would tend to increase the friction angle of the granular soil in the vicinity of the driven pile. Large-diameter piles would tend to displace and densify more soil than smaller-diameter piles.

Friction Pile for Granular Soil. As the name implies, a friction pile develops its load carrying capacity due to the frictional resistance between the granular soil and the pile perimeter. Piles subjected to vertical uplift forces would be designed as friction piles because there would be no end-bearing resistance as the pile is pulled from the ground.

Based on a linear increase in frictional resistance with confining pressure, the average ultimate frictional capacity (q_{ult}) can be calculated as follows:

$$q_{\text{ult}} = \frac{Q_s}{\text{surface area}} = \sigma'_h \tan \phi_w = \sigma'_v\, k \tan \phi_w \tag{6.27}$$

where q_{ult} = the average ultimate frictional capacity for the pile or pier
Q_s = ultimate skin friction resistance force
Surface area = perimeter surface area of the pile, which is equal to $4DL$ for a square pile and πDL for a round pile (D = diameter or width of pile and L = length of pile)
σ'_h = average horizontal effective stress over the length of the pile or pier
σ'_v = average vertical effective stress over the length of the pile or pier
k = dimensionless parameter equal to σ'_h divided by σ'_v (because of the densification of the granular soil associated with driven displacement piles, values of k between 1 and 2 are often assumed)
ϕ_w = friction angle between the cohesionless soil and the perimeter of the pile or pier (degrees)

Commonly used friction angles are $\phi_w = \frac{3}{4}\phi$ for wood and concrete piles and $\phi_w = 20°$ for steel piles.

In Eq. (6.27), the term $\sigma'_h \tan \phi_w$ equals the shear strength (τ_f) between the pile or pier surface and the granular soil. This term is identical to Eq. (6.9) (with $c' = 0$), that is, $\tau_f = \sigma'_n \tan \phi'$. Thus, the frictional resistance force (Q_s) in Eq. (6.27) is equal to the perimeter surface area times the shear strength of the soil at the pile or pier surface.

Combined End-Bearing and Friction Pile in Granular Soil. Piles and piers subjected to vertical compressive loads and embedded in a deposit of granular soil are usually treated in the design analysis as combined end-bearing and friction piles or piers. This is because the pile or pier can develop substantial load-carrying capacity from both end-bearing and frictional resistance. To calculate the ultimate pile or pier capacity for a condition of combined end-bearing and friction, the value of Q_p from Eq. (6.26) is added to the value of Q_s from Eq. (6.27). Usually the ultimate capacity is divided by a factor of safety of 3 in order to calculate the allowable pile or pier load.

Pile Groups in Granular Soil. The previous discussion dealt with the load capacity of a single pile in cohesionless soil. Usually pile groups are used to support the foundation elements, such as a group of piles supporting a pile cap or a mat slab. In loose sand and gravel deposits, the load-carrying capacity of each pile in the group may be greater than that of a single pile because of the densification effect due to driving the piles. Because of this densification effect, the load capacity of the group is often taken as the load capacity of a single pile times the number of piles in the group. An exception would be a situation where a weak layer underlies the cohesionless soil. In this case, group action of the piles could cause them to punch through the granular soil and into the weaker layer or cause excessive settlement of the weak layer located below the pile tips.

In order to determine the settlement of the strata underlying the pile group, the 2:1 approximation (see Art. 6.4.2) can be used to determine the increase in vertical stress ($\Delta\sigma_v$) for those soil layers located below the pile tip. If the piles in the group are principally end-bearing, then the 2:1 approximation starts at the tip of the piles (L = bottom length of the pile group, B = width of the pile group, and z = depth below the tip of the piles, see Eq. 6.16). If the pile group develops its load-carrying capacity principally through side friction, then the 2:1 approximation starts at a depth of $\frac{2}{3}D$, where D = depth of the pile group.

6.6.3 Bearing Capacity for Deep Foundations in Cohesive Soil

The load-carrying capacity of piles and piers in cohesive soil is more complex than the analysis for granular soil. Some of the factors that may need to be considered in the analysis are as follows (AASHTO, "Standard Specifications for Bridges," 16th ed., American Association of State Highway and Transportation Officials, Washington, DC):

- A lower load-carrying capacity of a pile in a pile group as compared to that of a single pile.
- The settlement of the underlying cohesive soil due to the load of the pile group.
- The effects of driving piles on adjacent structures or slopes. The ground will often heave around piles driven into soft and saturated cohesive soil.
- The increase in load on the pile due to negative skin friction (i.e., down-drag loads) from consolidating soil.
- The effects of uplift loads from expansive and swelling clays.
- The reduction in shear strength of the cohesive soil due to construction techniques, such as the disturbance of sensitive clays or development of excess pore water pressures during the driving of the pile. There is often an increase in load-carrying capacity of a pile after it has been driven into a soft and saturated clay deposit. This increase with time is known as **freeze** or **setup** and is caused primarily by the dissipation of excess pore water pressures.
- The influence of fluctuations in the elevation of the groundwater table on the load-carrying capacity when analyzed in terms of effective stresses.

Total Stress Analysis. The ultimate load capacity of a single pile or pier in cohesive soil is often determined by performing a total stress analysis. This is because the critical load on the pile, such as from wind or earthquake loads, is a short-term loading condition and thus the undrained shear strength of the cohesive soil will govern. The total stress analysis for a single pile or pier in cohesive soil typically is based on the undrained shear strength ($s_u = c$) of the cohesive soil.

The ultimate load capacity of the pile or pier in cohesive soil would equal the sum of the ultimate end-bearing and ultimate side adhesion components. Using the Terzaghi bearing capacity equation (Eq. 6.24), the ultimate load capacity (Q_{ult}) of a single pile or pier in cohesive soil equals:

$$Q_{\text{ult}} = \text{end bearing} + \text{side adhesion}$$

$$= cN_c\ (\text{area of tip}) + c_A\ (\text{surface area})$$

or

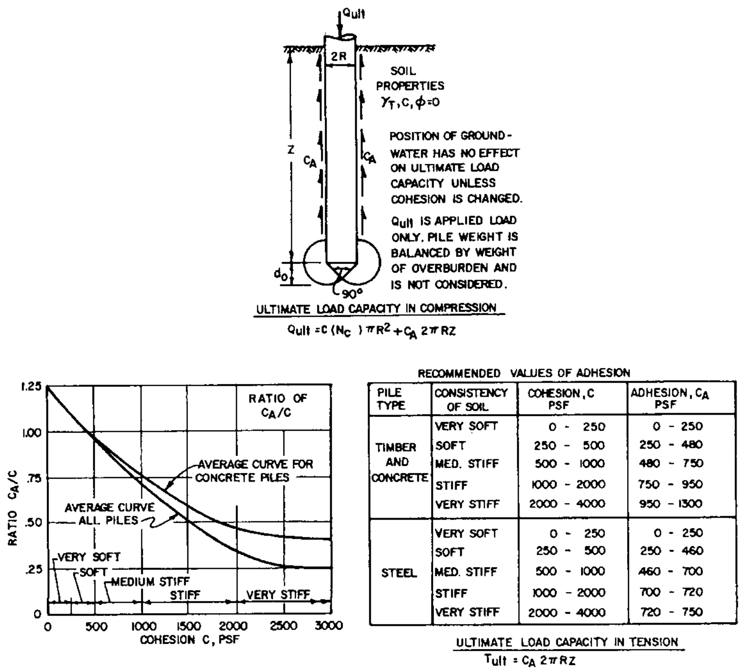
$$Q_{\text{ult}} = c9(\pi R^2) + c_A(2\pi RL)$$

$$= 9c\pi R^2 + 2\pi c_A RL \tag{6.28}$$

where Q_{ult} = ultimate load capacity of the pile or pier
c = cohesion of the cohesive soil at the pile tip (because it is a total stress analysis, the undrained shear strength ($s_u = c$) is often used)
R = radius of the pile or pier
L = length of the embedment of the pile
c_A = adhesion between the cohesive soil and pile or pier perimeter

For Eq. (6.28), the usual assumption is $N_c = 9$. Figure 6.31 can be used to determine the value of the adhesion (c_A) for different types of piles and cohesive soil conditions. If the pile or pier is subjected to an uplift force, then the first term in Eq. (6.28) is set equal to zero. Usually the ultimate capacity is divided by a factor of safety of 3 in order to calculate the allowable pile or pier load.

Pile Groups. The bearing capacity of pile groups in cohesive soils is normally less than the sum of individual piles in the group, and this reduction in group



RECOMMENDED VALUES OF ADHESION

PILE TYPE	CONSISTENCY OF SOIL	COHESION, C PSF	ADHESION, C_A PSF
TIMBER AND CONCRETE	VERY SOFT	0 - 250	0 - 250
	SOFT	250 - 500	250 - 480
	MED. STIFF	500 - 1000	480 - 750
	STIFF	1000 - 2000	750 - 950
	VERY STIFF	2000 - 4000	950 - 1300
STEEL	VERY SOFT	0 - 250	0 - 250
	SOFT	250 - 500	250 - 460
	MED. STIFF	500 - 1000	460 - 700
	STIFF	1000 - 2000	700 - 720
	VERY STIFF	2000 - 4000	720 - 750

ULTIMATE LOAD CAPACITY IN TENSION
$T_{ult} = C_A 2\pi RZ$

FIGURE 6.31 Ultimate capacity for a single pile or pier in cohesive soil. (*Reproduced from NAVFAC DM-7.2, 1982.*)

capacity must be considered in the analysis. The "group efficiency" is defined as the ratio of the ultimate load capacity of each pile in the group to the ultimate load capacity of a single isolated pile. If the spacing between piles in the group is at a distance that is greater than about 7 times the pile diameter, then the group efficiency is equal to 1 (i.e., no reduction in pile capacity for group action). The group efficiency decreases as the piles become closer together in the pile group. Figure 6.32 can be used to determine the ultimate load capacity of a pile group in cohesive soil.

Similar to pile groups in cohesionless soil, the settlement of the strata underlying the pile group can be evaluated by using the 2:1 approximation (see Art. 6.4.2) to calculate the increase in vertical stress ($\Delta\sigma_v$) for those soil layers located below the pile tip. If the piles in the group develop their load-carrying capacity principally by end-bearing in cohesive soil, then the 2:1 approximation starts at the tip of the piles (L = bottom length of the pile group, B = width of the pile group, and z = depth below the tip of the piles, see Eq. 6.16). If the pile group develops its load-carrying capacity principally through cohesive soil adhesion along the pile perimeter, then the 2:1 approximation starts at a depth of ⅔ D, where D = depth of the pile group.

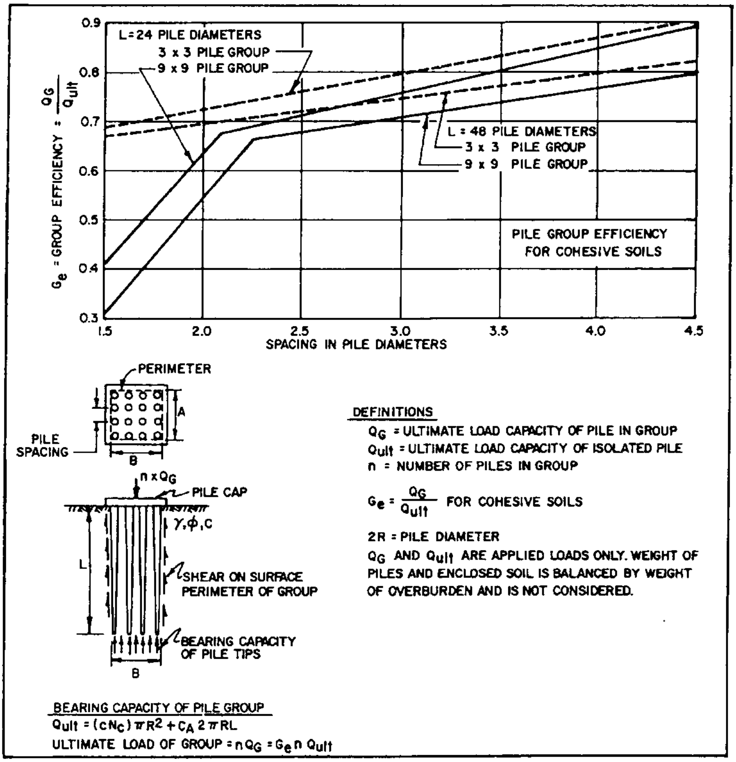


FIGURE 6.32 Ultimate capacity of a pile group in cohesive soil. (*Developed by Whitaker, reproduced from NAVFAC DM-7.2, 1982.*)

6.7 RETAINING WALLS

A **retaining wall** is defined as a structure whose primary purpose is to provide lateral support for soil or rock. In some cases, such as basement walls and certain types of bridge abutments, it may also support vertical loads. The more common types of retaining walls are shown in Fig. 6.33 and include **gravity walls, cantilevered walls, counterfort walls,** and **crib walls.** Gravity retaining walls are routinely built of plain concrete or stone, and the wall depends primarily on its massive

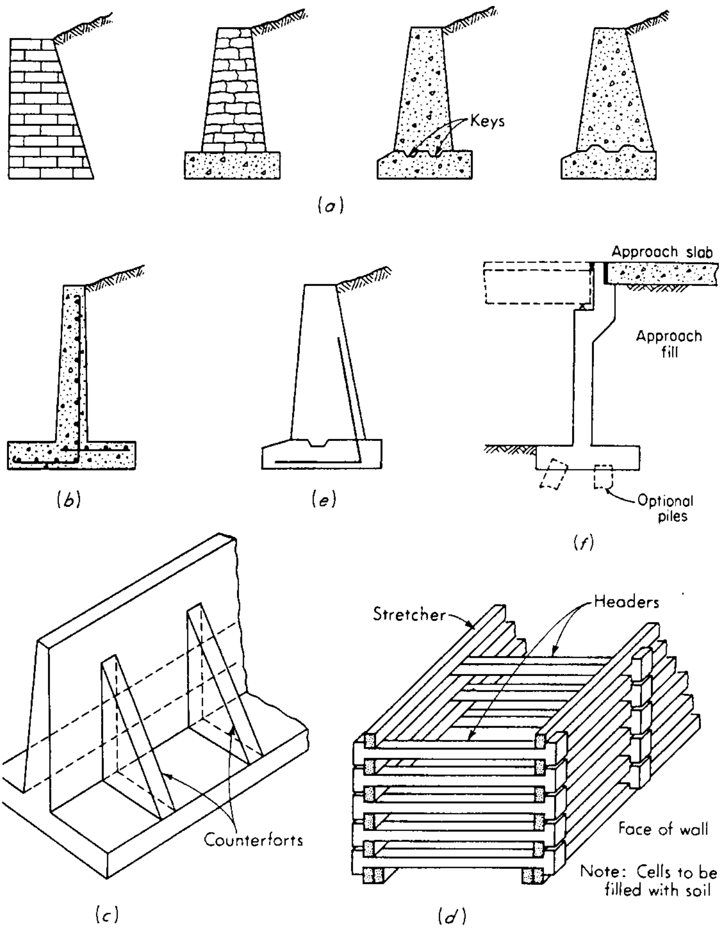


FIGURE 6.33 Common types of retaining walls: (*a*) Gravity walls of stone, brick, or plain concrete. Weight provides overturning and sliding stability; (*b*) cantilevered wall; (*c*) counterfort, or buttressed, wall. If backfill covers counterforts, the wall is termed a counterfort retaining wall; (*d*) crib wall; (*e*) semigravity wall (often steel reinforcement is used); (*f*) bridge abutment. (*Reproduced from J. E. Bowles, "Foundation Analysis and Design," 3d ed., McGraw-Hill Publishing Co., New York, with permission of McGraw-Hill, Inc.*)

weight to resist failure from overturning and sliding. Counterfort walls consist of a footing, a wall stem, and intermittent vertical ribs (called counterforts) that tie the footing and wall stem together. Crib walls consist of interlocking concrete members that form cells which are then filled with compacted soil.

Granular soils (sands or gravels) are the standard recommendation for backfill material. There are several reasons for this recommendation:

1. **Predictable Behavior.** Import granular backfill generally has a more predictable behavior in terms of earth pressure exerted on the wall. If silts or clays are used as backfill material, expansive soil-related forces could be generated by these soil types.
2. **Drainage System.** To prevent the build-up of hydrostatic water pressure on the retaining wall, a drainage system is often constructed at the heel of the wall. This system will be more effective if highly permeable granular soil is used as backfill.
3. **Frost Action.** In cold climates, the formation of ice lenses in the backfill soil can cause so much lateral movement that the retaining wall will become unusable. Backfill soil consisting of granular soil and the installation of a drainage system at the heel of the wall will help to protect the wall from frost action.

6.7.1 Retaining Wall Analyses

Figure 6.34 shows various types of retaining walls and the soil pressures acting on the walls. Three types of soil pressures act on a retaining wall: (1) active earth pressure, which is exerted on the back side of the wall, (2) passive earth pressure, which acts on the front of the retaining wall footing, and (3) bearing pressure, which acts on the bottom of the retaining wall footing. These three pressures are individually discussed below.

Active Earth Pressure. In order to calculate the active earth pressure resultant force (P_A), in kN per linear meter of wall or pounds per linear foot of wall, the following equation is used for granular backfill:

$$P_A = \tfrac{1}{2} k_A \gamma_t H^2 \tag{6.29}$$

where k_A = active earth pressure coefficient
γ_t = total unit weight of the granular backfill
H = height over which the active earth pressure acts as defined in Fig. 6.34*a*

In its simplest form, the active earth pressure coefficient (k_A) is equal to:

$$k_A = \tan^2(45° - \tfrac{1}{2}\phi) \tag{6.30}$$

where ϕ = friction angle of the granular backfill. Equation (6.30) is known as the active Rankine state, after the British engineer Rankine, who in 1857 obtained this relationship. Equation (6.30) is valid only for the simple case of a retaining wall that has a vertical rear face, no friction between the rear wall face and backfill soil, and the backfill ground surface is horizontal. For retaining walls that do not meet these requirements, the active earth pressure coefficient (k_A) for Eq. (6.29) is often determined using the Coulomb equation (see Fig. 6.35). Often the wall friction is neglected ($\delta = 0°$), but if it is included in the analysis, typical values are $\delta = \tfrac{3}{4}\phi$

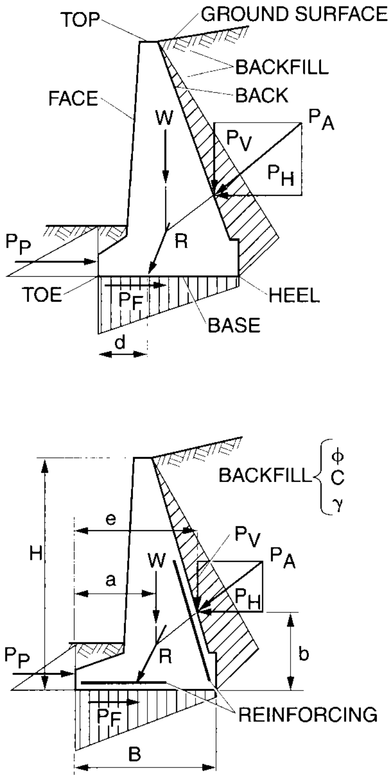


FIGURE 6.34a Gravity and semigravity retaining walls. (*From NAVFAC DM-7.2, 1982.*)

for the wall friction between granular soil and wood or concrete walls and $\delta = 20°$ for the wall friction between granular soil and steel walls such as sheet-pile walls. Note in Fig. 6.35 that when wall friction angle δ is used in the analysis, the active earth pressure resultant force (P_A) is inclined at an angle equal to δ.

Additional important details concerning the active earth pressure are as follows:

1. **Sufficient Movement.** There must be sufficient movement of the retaining wall in order to develop the active earth pressure of the backfill. For dense granular soil, the amount of wall translation to reach the active earth pressure state is usually very small (i.e., to reach active state, wall translation $\geq 0.0005\,H$, where H = height of wall).
2. **Triangular Distribution.** As shown in Figs. 6.34 and 6.35, the active earth pressure is a triangular distribution and thus the active earth pressure resultant force (P_A) is located at a distance equal to ⅓H above the base of the wall.
3. **Surcharge Pressure.** If there is a uniform surcharge pressure (Q) acting upon the entire ground surface behind the wall, then there would be an additional horizontal pressure exerted upon the retaining wall equal to the product of k_A

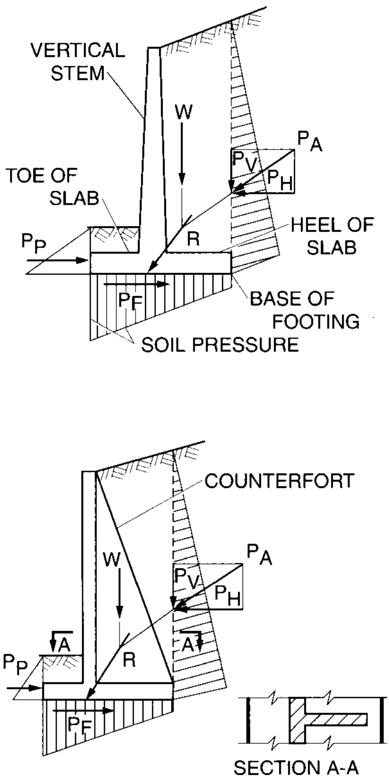


FIGURE 6.34b Cantilever and counterfort retaining walls. (*From NAVFAC DM-7.2, 1982.*)

times Q. Thus, the resultant force (P_2), in kN per linear m of wall or lb per linear ft of wall, acting on the retaining wall due to the surcharge (Q) is equal to $P_2 = QHk_A$, where Q = uniform vertical surcharge acting upon the entire ground surface behind the retaining wall, k_A = active earth pressure coefficient (Eq. (6.30) or Fig. 6.35), and H = height of the retaining wall. Because this pressure acting upon the retaining wall is uniform, the resultant force (P_2) is located at midheight of the retaining wall.

4. **Active Wedge:** The active wedge is defined as that zone of soil involved in the development of the active earth pressures upon the wall. This active wedge must move laterally in order to develop the active earth pressures. It is important that building footings or other load-carrying members are not supported by the active wedge, or else they will be subjected to lateral movement. The active wedge is inclined at an angle of $45° + \phi/2$ from the horizontal.

Passive Earth Pressure. As shown in Fig. 6.34, the passive earth pressure is developed along the front side of the footing. Passive pressure is developed when the wall footing moves laterally into the soil and a passive wedge is developed. In

LOCATION OF RESULTANT

MOMENTS ABOUT TOE:

$$d = \frac{Wa + P_V e - P_H b}{W + P_V}$$

ASSUMING $P_P = 0$

OVERTURNING

MOMENTS ABOUT TOE:

$$F = \frac{Wa}{P_H b - P_V e} \geqq 1.5$$

IGNORE OVERTURNING IF R IS WITHIN MIDDLE THIRD (SOIL), MIDDLE HALF (ROCK).
CHECK R AT DIFFERENT HORIZONTAL PLANES FOR GRAVITY WALLS.

RESISTANCE AGAINST SLIDING

$$F = \frac{(W + P_V)\ \mathrm{TAN}\ \delta + C_a B}{P_H} \geqq 1.5$$

$$F = \frac{(W+P_V)\ \mathrm{TAN}\ \delta + C_a B + P_P}{P_H} \geqq 2.0$$

$$P_F = (W + P_V)\ \mathrm{TAN}\ \delta + C_a B$$

C_a = ADHESION BETWEEN SOIL AND BASE

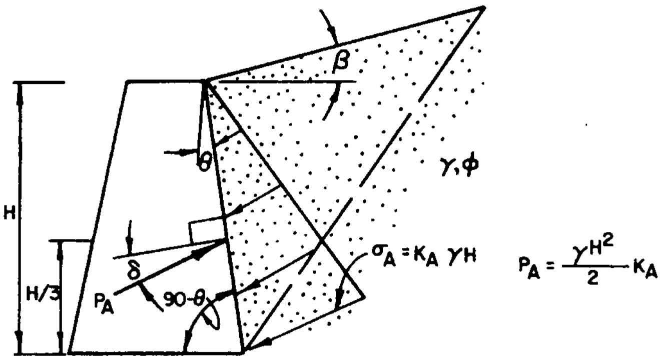
TAN δ = FRICTION FACTOR BETWEEN SOIL AND BASE

W = INCLUDES WEIGHT OF WALL AND SOIL IN FRONT FOR GRAVITY AND SEMIGRAVITY WALLS. INCLUDES WEIGHT OF WALL AND SOIL ABOVE FOOTING, FOR CANTILEVER AND COUNTERFORT WALLS.

FIGURE 6.34c Design analysis for retaining walls shown in Figs. 6.34*a* and 6.34*b*. (*From NAVFAC DM-7.2, 1982.*)

order to calculate the passive resultant force (P_p), the following equation is used assuming that there is cohesionless soil in front of the wall footing:

$$P_p = \tfrac{1}{2} k_p \gamma_t D^2 \tag{6.31}$$



$$K_A = \frac{\cos^2(\phi - \theta)}{\cos^2\theta \cos(\theta + \delta)\left[1 + \sqrt{\frac{\sin(\phi+\delta)\sin(\phi-\beta)}{\cos(\theta+\delta)\cos(\theta-\beta)}}\right]^2}$$

FIGURE 6.35 Coulomb's earth pressure (k_A) equation. (*From NAVFAC DM-7.2, 1982.*)

where P_p = passive resultant force in kN per linear m of wall or lb per linear ft of wall
k_p = passive earth pressure coefficient
γ_t = total unit weight of the soil located in front of the wall footing
D = depth of the wall footing (vertical distance from the ground surface in front of the retaining wall to the bottom of the footing)

The passive earth pressure coefficient (k_p) is equal to:

$$k_p = \tan^2(45° + ½\phi) \tag{6.32}$$

where ϕ = friction angle of the soil in front of the wall footing. Equation (6.32) is known as the passive Rankine state. In order to develop passive pressure, the wall footing must more laterally into the soil. The wall translation to reach the passive state is at least twice that required to reach the active earth pressure state. Usually it is desirable to limit the amount of wall translation by applying a reduction factor to the passive pressure. A commonly used reduction factor is 2.0. The soil engineer routinely reduces the passive pressure by ½ (reduction factor = 2.0) and then refers to the value as the allowable passive pressure.

Footing Bearing Pressure. In order to calculate the footing bearing pressure, the first step is to sum the vertical loads, such as the wall and footing weights. The vertical loads can be represented by a single resultant vertical force, per linear m or ft of wall, that is offset by a distance (eccentricity) from the toe of the footing. This can then be converted to a pressure distribution by using Eq. (6.25). The largest bearing pressure is routinely at the toe of the footing and it should not exceed the allowable bearing pressure (Art. 6.6.1).

Retaining Wall Analyses. Once the active earth pressure resultant force (P_A) and the passive resultant force (P_p) have been calculated, the design analysis is performed as indicated in Fig. 6.34*c*. The retaining wall analysis includes determining the resultant location of the forces (i.e., calculate d, which should be within the middle third of the footing), the factor of safety for overturning, and the factor of safety for sliding. The adhesion (c_a) between the bottom of the footing and the underlying soil is often ignored for the sliding analysis.

6.7.2 Restrained Retaining Walls

As mentioned in the previous article, in order for the active wedge to be developed, there must be sufficient movement of the retaining wall. There are many cases where movement of the retaining wall is restricted. Examples include massive bridge abutments, rigid basement walls, and retaining walls that are anchored in nonyielding rock. These cases are often described as **restrained retaining walls.**

In order to determine the earth pressure acting on a restrained retaining wall, Eq. (6.29) can be utilized where the coefficient of earth pressure at rest (k_0) is substituted for k_A. A common value of k_0 for granular soil that is used for restrained retaining walls is 0.5. Restrained retaining walls are especially susceptible to higher earth pressures induced by heavy compaction equipment, and extra care must be taken during the compaction of backfill for restrained retaining walls.

6.7.3 Mechanically Stabilized Earth Retaining Walls

Mechanically stabilized earth retaining walls (also known as **MSE retaining walls**) are typically composed of strip- or grid-type (geosynthetic) reinforcement. Because they are often more economical to construct than conventional concrete retaining walls, mechanically stabilized earth retaining walls have become very popular in the past decade. A mechanically stabilized earth retaining wall is composed of three elements: (1) wall facing material, (2) soil reinforcement, such as strip- or grid-type reinforcement, and (3) compacted fill between the soil reinforcement.

The design analysis for a mechanically stabilized earth retaining wall is more complex than for a cantilevered retaining wall. For a mechanically stabilized earth retaining wall, both the internal and external stability must be checked.

External Stability. The analysis for the external stability is similar to that for a gravity retaining wall. For example, Figs. 6.36 and 6.37 present the design analysis for external stability for a level backfill condition and a sloping backfill condition. In both Figs. 6.36 and 6.37, the zone of mechanically stabilized earth mass is treated in a similar fashion as a massive gravity retaining wall. The following analyses must be performed:

1. Allowable bearing pressure: the bearing pressure due to the reinforced soil mass must not exceed the allowable bearing pressure.
2. Factor of safety of sliding: the reinforced soil mass must have an adequate factor of safety for sliding.
3. Factor of safety of overturning; the reinforced soil mass must have an adequate factor of safety for overturning about Point O.

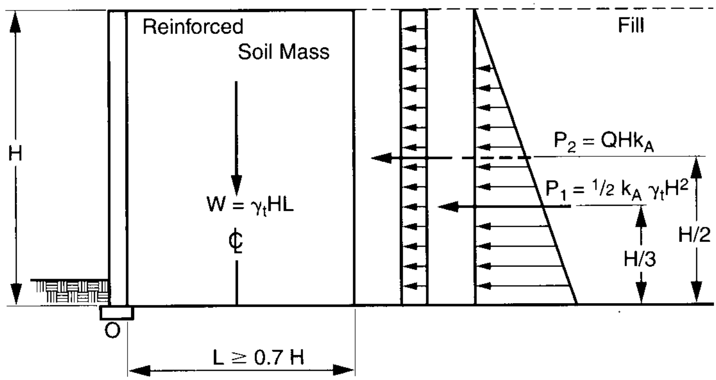


FIGURE 6.36 Design analysis for mechanically stabilized earth retaining wall having horizontal backfill. (*Adapted from AASHTO, "Standard Specifications for Highway Bridges," 16th ed., American Association of State Highway and Transportation Officials, Washington, DC.*)

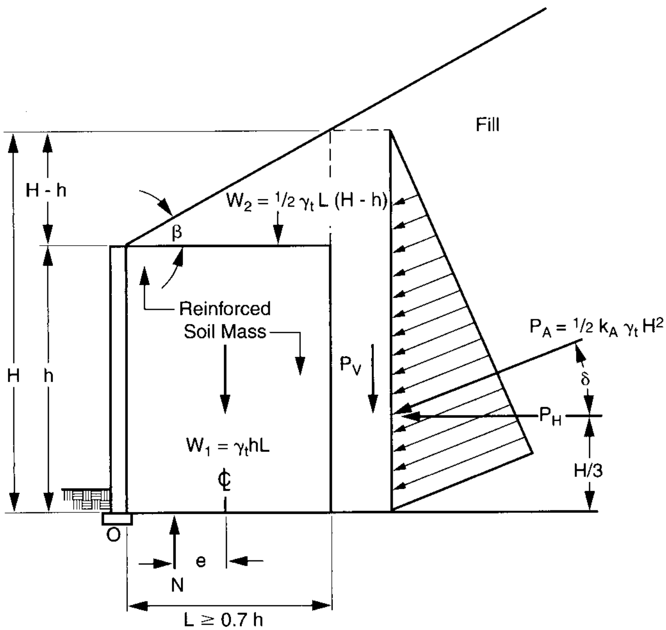


FIGURE 6.37 Design analysis for mechanically stabilized earth retaining wall having sloping backfill. (*Adapted from AASHTO, "Standard Specifications for Highway Bridges," 16th ed., American Association of State Highway and Transportation Officials, Washington, DC.*)

4. Resultant of vertical forces: the resultant of the vertical forces N must be within the middle ⅓ of the base of the reinforced soil mass.
5. Stability of reinforced soil mass: the stability of the entire reinforced soil mass (i.e., shear failure below the bottom of the wall) would have to be checked.

Note in Figure 6.36 that two forces (P_1 and P_2) are shown acting on the reinforced soil mass. The first force (P_1) is determined from the standard active earth pressure resultant equation (i.e., Eq. 6.29). The second force (P_2) is due to a uniform surcharge (Q) applied to the entire ground surface behind the mechanically stabilized earth retaining wall. If the wall does not have a surcharge, then P_2 is equal to zero.

Figure 6.37 presents the active earth pressure force for an inclined slope behind the retaining wall. Note in Fig. 6.37 that the friction (δ) of the soil along the back side of the reinforced soil mass has been included in the analysis. The value of k_A would be obtained from Coulomb's earth pressure equation (Fig. 6.35). As a conservative approach, the friction angle (δ) can be assumed to be equal to zero and then $P_H = P_A$. Note in both Figs. 6.36 and 6.37 that the minimum width of the reinforced soil mass must be at least 7⁄10 the height of the reinforced soil mass.

Internal Stability. To check the stability of the mechanically stabilized zone, a slope stability analysis can be performed where the soil reinforcement is modeled as horizontal forces equivalent to its allowable tensile resistance. In addition to calculation of the factor of safety, the pull-out resistance of the reinforcement along the slip surface should also be checked.

The analysis of mechanically stabilized earth retaining walls is based on active earth pressures. It is assumed that the wall will move enough to develop the active wedge. As with concrete retaining walls, it is important that building footings or other load carrying members are not supported by the mechanically stabilized earth retaining wall and the active wedge, or else they could be subjected to lateral movement.

6.7.4 Sheet Pile Walls

Sheet pile retaining walls are widely used for waterfront construction and consist of interlocking members that are driven into place. Individual sheet piles come in many different sizes and shapes. Sheet piles have an interlocking joint that enables the individual segments to be connected together to form a solid wall.

Many different types of design methods are used for sheet pile walls. Figure 6.38 shows the most common type of design method. In Fig. 6.38, the term H represents the unsupported face of the sheet pile wall. As indicated in Fig. 6.38, this sheet pile wall is being used as a waterfront retaining structure and the level of water in front of the wall is at the same elevation as the groundwater table elevation behind the wall. For highly permeable soil, such as clean sand and gravel, this often occurs because the water can quickly flow underneath the wall in order to equalize the water levels.

In Fig. 6.38, the term D represents that portion of the sheet pile wall that is anchored in soil. Also shown in Fig. 6.38 is a force designated as A_p. This represents a restraining force on the sheet pile wall due to the construction of a tieback, such as by using a rod that has a grouted end or is attached to an anchor block. Tieback anchors are often used in sheet pile wall construction in order to reduce the bending

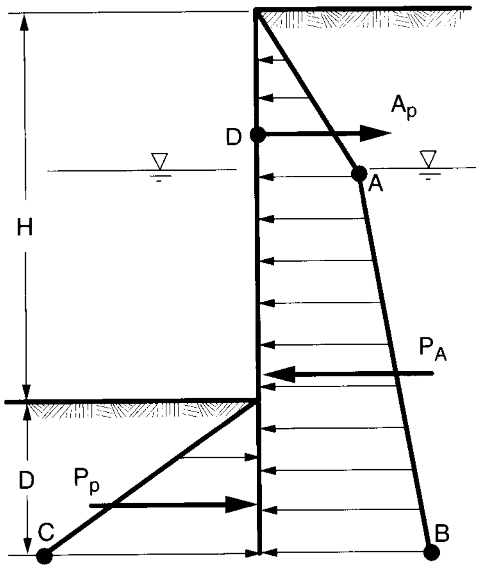


FIGURE 6.38 Earth pressure diagram for design of sheet pile wall. (*From NAVFAC DM-7.2, 1982.*)

moments in the sheet pile. When tieback anchors are used, the sheet pile wall is typically referred to as an **anchored bulkhead,** while if no tiebacks are utilized, the wall is called a **cantilevered sheet pile wall.**

Sheet pile walls tend to be relatively flexible. Thus, as indicated in Fig. 6.38, the design is based on active and passive earth pressures. For this analysis, a unit length (1 m or 1 ft) of sheet pile wall is assumed. The soil behind the wall is assumed to exert an active earth pressure on the sheet pile wall. At the groundwater table (Point A), the active earth pressure is equal to $k_A \gamma_t d_1$, where k_A = active earth pressure coefficient from Eq. (6.30) (the friction between the sheet pile wall and the soil is usually neglected in the design analysis), γ_t = total unit weight of the soil above the groundwater table, and d_1 = depth from the ground surface to the groundwater table. At Point B in Fig. 6.38, the active earth pressure equals $k_A \gamma_t d_1 + k_A \gamma_b d_2$, where γ_b = buoyant unit weight of the soil below the groundwater table and d_2 = depth from the groundwater table to the bottom of the sheet pile wall. For a sheet pile wall having assumed values of H and D (see Fig. 6.38), and using the calculated values of active earth pressure at Points A and B, the active earth pressure resultant force (P_A), in kN per linear m of wall or lb per linear foot of wall, can be calculated.

The soil in front of the wall is assumed to exert a passive earth pressure on the sheet pile wall. The passive earth pressure at Point C in Fig. 6.38 is equal to $k_p \gamma_b D$, where the passive earth pressure coefficient (k_p) can be calculated from Eq. (6.32). Similar to the analysis of cantilever retaining walls, if it is desirable to limit the amount of sheet pile wall translation, then a reduction factor can be applied to the passive pressure. Once the allowable passive pressure is known at Point C, the passive resultant force (P_p) can be readily calculated. As an alternative solution for the passive pressure, Eq. (6.31) can be used to calculate P_p with the buoyant unit weight (γ_b) substituted for the total unit weight (γ_t) and the depth D as shown in Fig. 6.38.

Note that a water pressure has not been included in the analysis. This is because the water level is the same on both sides of the wall and water pressure cancels itself out. However, if the water level was higher behind than in front of the wall, then water pressure forces would be generated behind the wall.

The design of sheet pile walls requires the following analyses: (1) evaluation of the earth pressure resultant forces P_A and P_p as previously described, (2) determination of the required depth D of piling penetration, (3) calculation of the maximum bending moment ($M_{\max}$) which is used to determine the maximum stress in the sheet pile, and (4) selection of the appropriate piling type, size, and construction details.

A typical design process is to assume a depth D (Fig. 6.38) and then calculate the factor of safety for toe failure (i.e., toe kick-out) by the summation of moments at the tieback anchor (Point D). The factor of safety is defined as the moment due to the passive force divided by the moment due to the active force. Values of acceptable factor of safety for toe failure are 2 to 3.

Once the depth D of the sheet pile wall is known, the anchor pull (A_p) must be calculated. The anchor pull is determined by the summation of forces in the horizontal direction, or: $A_p = P_A - P_p/F$, where P_A and P_p are the resultant active and passive forces and F is the factor of safety that was obtained from the toe failure analysis. Based on the earth pressure diagram (Fig. 6.38) and the calculated value of A_p, elementary structural mechanics can be used to determine the maximum moment in the sheet pile wall. The maximum moment divided by the section modulus can then be compared with the allowable design stresses.

6.7.5 Temporary Retaining Walls

Temporary retaining walls are often used during construction, such as for the support of the sides of an excavation that is made below-grade in order to construct the building foundation. If the temporary retaining wall has the ability to develop the active wedge, then the basic active earth pressure principles described in the previous sections can be used for the design of the temporary retaining walls.

Especially in urban areas, movement of the temporary retaining wall may have to be restricted to prevent damage to adjacent property. If movement of the retaining wall is restricted, the earth pressures will typically be between the active (k_A) and at-rest (k_0) values.

For some projects, the temporary retaining wall may be constructed of sheeting (such as sheet piles) that are supported by **horizontal braces,** also known as **struts.** Near or at the top of the temporary retaining wall, the struts restrict movement of the retaining wall and prevent the development of the active wedge. Because of this inability of the retaining wall to deform at the top, earth pressures near the top of the wall are in excess of the active (k_A) pressures. At the bottom of the wall, the soil is usually able to deform into the excavation, which results in a reduction in earth pressure, and the earth pressures at the bottom of the excavation tend to be constant or even decrease as shown in Fig. 6.39.

The earth pressure distributions shown in Fig. 6.39 were developed from actual measurements of the forces in struts during the construction of braced excavations. In Fig. 6.39, case *a* shows the earth pressure distribution for braced excavations in sand and cases *b* and *c* show the earth pressure distribution for clays. In Fig. 6.39,

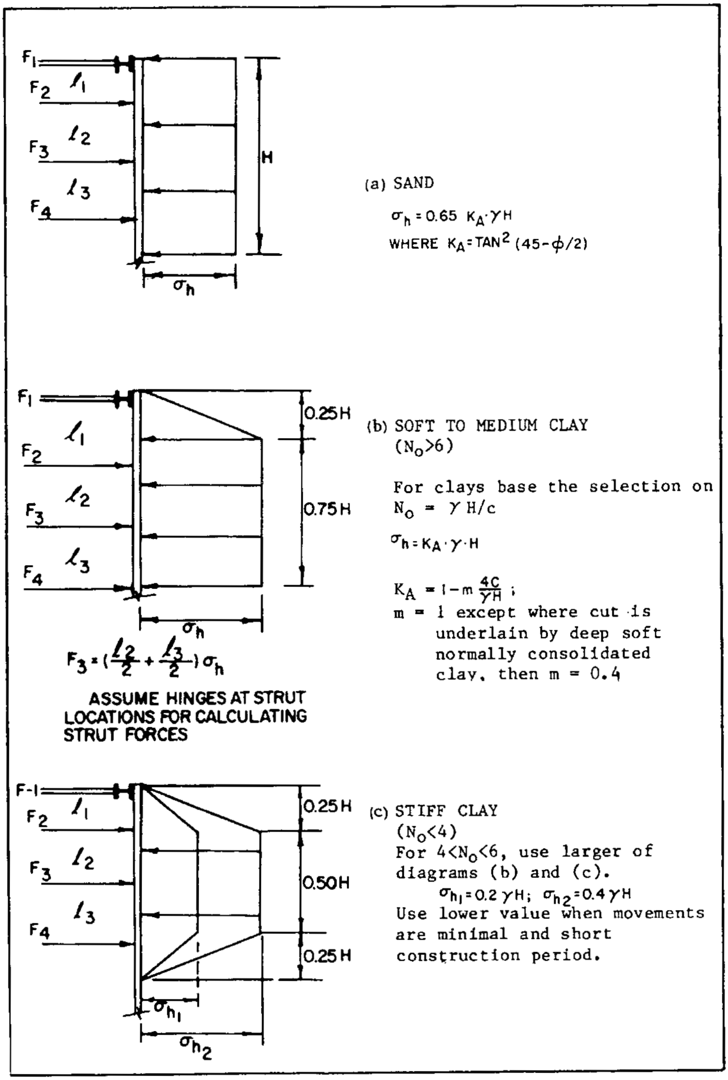


FIGURE 6.39 Earth pressure distribution on temporary braced walls. (*From NAVFAC DM-7.2 1982, originally developed by K. Terzaghi and R. B. Peck, "Soil Mechanics in Engineering Practice," 2d ed., John Wiley & Sons, Inc., New York.*)

the distance H represents the depth of the excavation (i.e., the height of the exposed wall surface). The earth pressure distribution is applied over the exposed height (H) of the wall surface with the earth pressures transferred from the wall sheeting to the struts (the struts are labeled with the forces F_1, F_2, etc.).

Any surcharge pressures, such as surcharge pressures on the ground surface adjacent the excavation, must be added to the pressure distributions shown in Fig. 6.39. In addition, if the sand deposit has a groundwater table that is above the level of the bottom of the excavation, then water pressures must be added to the case *a* pressure distribution shown in Fig. 6.39.

Because the excavations are temporary (i.e., short-term condition), the undrained shear strength ($s_u = c$) is used for the analysis of the earth pressure distributions for clay. The earth pressure distributions for clay (i.e., cases *b* and *c*) are not valid for permanent walls or for walls where the groundwater table is above the bottom of the excavation.

6.8 FOUNDATIONS

This section deals with the selection of the type of foundation. The selection of a particular type of foundation is often based on a number of factors, such as:

1. **Adequate Depth.** It must have an adequate depth to prevent frost damage. For such foundations as bridge piers, the depth of the foundation must be sufficient to prevent undermining by scour.
2. **Bearing Capacity Failure.** The foundation must be safe against a bearing capacity failure (Art. 6.6).
3. **Settlement.** The foundation must not settle to such an extent that it damages the structure (Art. 6.5).
4. **Quality.** The foundation must be of adequate quality so that it is not subjected to deterioration, such as the sulfate attack of concrete footings.
5. **Adequate Strength.** The foundation must be designed with sufficient strength that it does not fracture or break apart under the applied superstructure loads. It must also be properly constructed in conformance with the design specifications.
6. **Adverse Soil Changes.** The foundation must be able to resist long-term adverse soil changes. An example is expansive soil (silts and clays), which could expand or shrink causing movement of the foundation and damage to the structure.
7. **Seismic Forces.** The foundation must be able to support the structure during an earthquake without excessive settlement or lateral movement.

6.8.1 Shallow Foundations

A shallow foundation is often selected when the structural load will not cause excessive settlement of the underlying soil layers. In general, shallow foundations are more economical to construct than deep foundations. Common types of shallow foundations are listed in Table 6.2 and described below:

1. **Spread Footings, Combined Footings, and Strip Footings.** These types of shallow foundations are probably the most common types of building foundations. Examples of these types of footings are shown in Fig. 6.40.
2. **Mat Foundation.** Examples of mat foundations are shown in Fig. 6.41. Based on economic considerations, mat foundations are constructed for the following reasons:
   (*a*) **Large Individual Footings.** A mat foundation is often constructed when the sum of individual footing areas exceeds about one-half of the total foundation area.
   (*b*) **Cavities or Compressible Lenses.** A mat foundation can be used when the subsurface exploration indicates that there will be unequal settlement caused by small cavities or compressible lenses below the foundation. A mat foundation would tend to span over the small cavities or weak lenses and create a more uniform settlement condition.
   (*c*) **Shallow Settlements.** A mat foundation can be recommended when shallow settlements predominate and the mat foundation would minimize differential settlements.
   (*d*) **Unequal Distribution of Loads.** For some structures, there can be a large difference in building loads acting on different areas of the foundation. Conventional spread footings could be subjected to excessive differential settle-

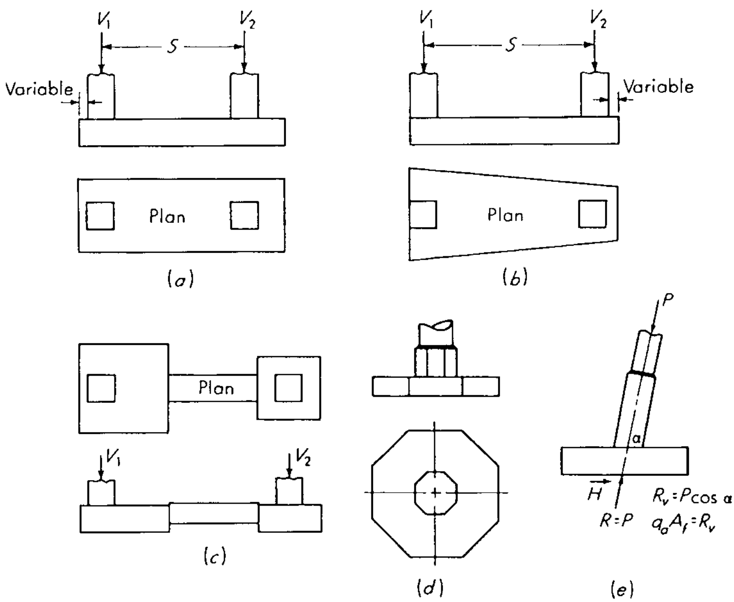


FIGURE 6.40 Examples of shallow foundations: (*a*) combined footing; (*b*) combined trapezoidal footing; (*c*) cantilever or strap footing; (*d*) octagonal footing; (*e*) eccentric loaded footing with resultant coincident with area so soil pressure is uniform. (*From J. E. Bowles, "Foundation Analysis and Design," 2d ed., McGraw-Hill Publishing Co., New York, with permission of McGraw-Hill, Inc.*)

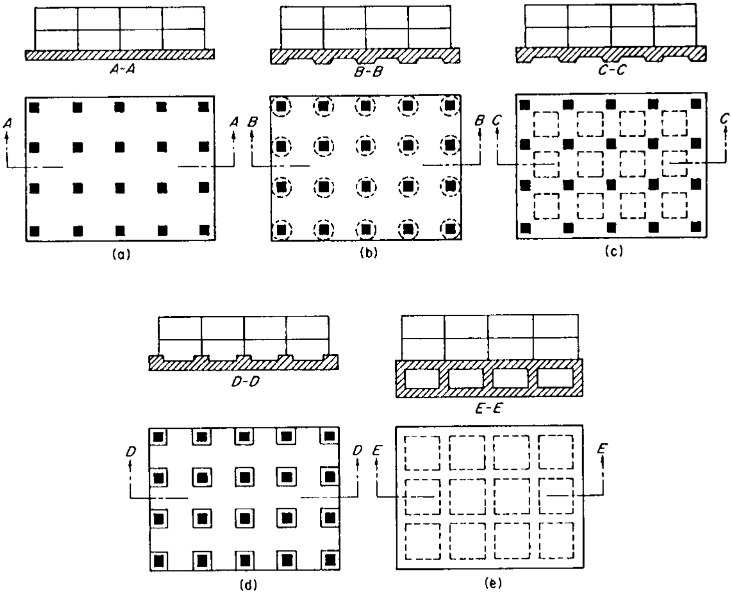


FIGURE 6.41 Examples of mat foundations: (*a*) Flat plate; (*b*) plate thickened under columns; (*c*) beam-and-slab; (*d*) plate with pedestals; (*e*) basement walls as part of mat. (*From J. E. Bowles, "Foundation Analysis and Design," 2d ed., McGraw-Hill Publishing Co., New York, with permission of McGraw-Hill, Inc.*)

ment, but a mat foundation would tend to distribute the unequal building loads and reduce the differential settlements.

(*e*) **Hydrostatic Uplift.** When the foundation will be subjected to hydrostatic uplift due to a high groundwater table, a mat foundation could be used to resist the uplift forces.

3. **Post-Tensioned Slabs-on-Grade.** Post-tensioned slabs-on-grade are common in southern California and other parts of the United States. They are an economical foundation type when there is no ground freezing or the depth of frost penetration is low. The most common uses of post-tensioned slabs-on-grade are to resist expansive soil forces or when the projected differential settlement exceeds the tolerable value for a conventional (lightly reinforced) slabs-on-grade. For example, post-tensioned slabs-on-grade are frequently recommended if the projected differential settlement is expected to exceed 2 cm (0.75 in). Installation and field inspection procedures for post-tensioned slabs-on-grade have been prepared by the Post-Tensioning Institute ("Design and Construction of Post-tensioned Slabs-on-Ground," 2d ed., Phoenix). Post-tensioned slabs-on-grade consists of concrete with embedded steel tendons that are encased in thick plastic sheaths. The plastic sheath prevents the tendon from coming in contact with the concrete and permits the tendon to slide within the hardened concrete during the tensioning operations. Usually tendons have a dead end (anchoring plate) in the perimeter (edge) beam and a stressing end at the opposite perimeter beam to enable the tendons to be stressed from one end. The Post-Tensioning Institute

("Design and Construction of Post-tensioned Slabs-on-Ground," 2d ed., Phoenix) provides typical anchorage details for the tendons.

4. **Shallow Foundation Alternatives.** If the expected settlement for a proposed shallow foundation is too large, then other options for foundation support or soil stabilization must be evaluated. Some commonly used alternatives are as follows:
   (*a*) **Grading.** Grading operations can be used to remove the compressible soil layer and replace it with structural fill. Usually the grading option is economical only if the compressible soil layer is near the ground surface and the groundwater table is below the compressible soil layer or the groundwater table can be economically lowered.
   (*b*) **Surcharge.** If the site contains an underlying compressible cohesive soil layer, the site can be surcharged with a fill layer placed at the ground surface. Vertical drains (such as wick drains or sand drains) can be installed in the compressible soil layer to reduce the drainage paths and speed up the consolidation process. Once the compressible cohesive soil layer has had sufficient consolidation, the fill surcharge layer is removed and the building is constructed.
   (*c*) **Densification of Soil.** Many different methods can be used to densify loose or soft soil. For example, vibro-flotation and dynamic compaction are often effective at increasing the density of loose sand deposits. Another option is compaction grouting, which consists of intruding a mass of very thick consistency grout into the soil, which both displaces and compacts the loose soil.
   (*d*) **Floating Foundation.** A floating foundation is a special type of deep foundation where the weight of the structure is balanced by the removal of soil and construction of an underground basement.

6.8.2 Deep Foundations

Probably the most common type of deep foundation is the pile foundation. Table 6.15 presents pile type characteristics and uses. Piles can consist of wood (timber), steel H-sections, precast concrete, cast-in-place concrete, pressure injected concrete, concrete filled steel pipe piles, and composite type piles. Piles are either driven into place or installed in predrilled holes. Piles that are driven into place are generally considered to be low displacement or high displacement, depending on the amount of soil that must be pushed out of the way as the pile is driven. Examples of low-displacement piles are steel H-sections and open-ended steel pipe piles that do not form a soil plug at the end. Examples of high-displacement piles are solid section piles, such as round timber piles or square precast concrete piles, and steel pipe piles with a closed end.

A cast-in-place pile is formed by making a hole in the ground and then filling the hole with concrete. As shown in Fig. 6.42, in its simplest form, the cast-in-place pile consists of an uncased hole that is filled with concrete. If the soil tends to cave into the hole, then a shell-type pile can be installed (see Fig. 6.42). This consists of driving a steel shell or casing into the ground. The casing may be driven with a mandrel, which is then removed, and the casing is filled with concrete. In other cases, the casing can be driven into place and then slowly removed as the hole is filled with concrete.

Figure 6.43 shows typical pile configurations.

TABLE 6.15 Typical Pile Characteristics and Uses

Pile type	Timber	Steel	Cast-in-place concrete piles (shells driven without mandrel)	Cast-in-place concrete piles (shells withdrawn)
Maximum length	35 m (115 ft)	Practically unlimited	45 m (150 ft)	36 m (120 ft)
Optimum length	9–20 m (30–65 ft)	12–50 m (40–160 ft)	9–25 m (30–80 ft)	8–12 m (25–40 ft)
Applicable material specifications	ASTM-D25 for piles; PI-54 for quality of creosote; C1-60 for creosote treatment (standards of American Wood Preserves Assoc.)	ASTM-A36 for structural sections ASTM-A1 for rail sections	ACI	ACI[a]
Recommended maximum stresses	Measured at midpoint of length: 4–6 MPa (600–900 psi) for cedar, western hemlock, Norway pine, spruce, and depending on code 5–8 MPa (700–1200 psi) for southern pine, Douglas fir, oak cypress, and hickory	f_s = 65 to 140 MPa (9–20 ksi) f_s = 0.35–0.5 f_y	0.33 f'_c; 0.4 f'_c if shell gauge $\leq$ 14; shell stress = 0.35 f_y if thickness of shell $\geq$ 3 mm	0.25–0.33f'_c
Maximum load for usual conditions	270 kN (60 kips)	Maximum allowable stress $\times$ cross section	900 kN (200 kips)	1300 kN (300 kips)
Optimum-load range	130–225 kN (30–50 kips)	350–1050 kN (80–240 kips)	450–700 kN (100–150 kips)	350–900 kN (80–200 kips)
Disadvantages	Difficult to splice Vulnerable to damage in hard driving Vulnerable to decay unless treated, when piles are intermittently submerged	Vulnerable to corrosion HP section may be damaged or deflected by major obstructions	Hard to splice after concreting Considerable displacement	Concrete should be placed in dry hole More than average dependence on quality of workmanship

TABLE 6.15 Typical Pile Characteristics and Uses (Continued)

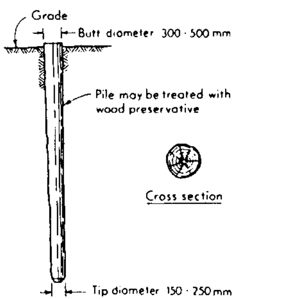
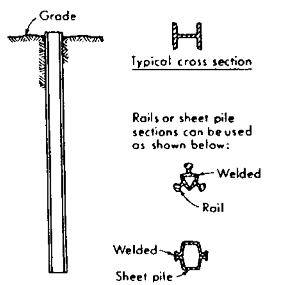
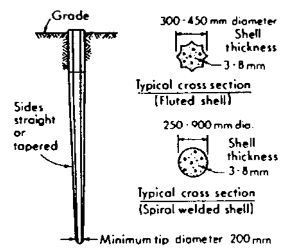
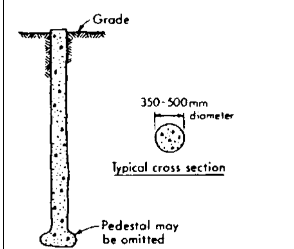
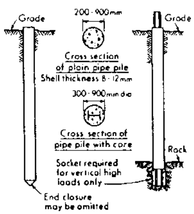
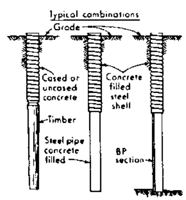
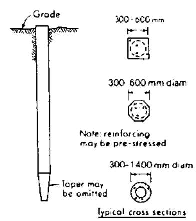
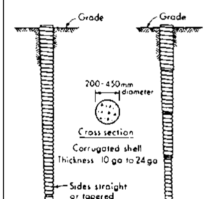
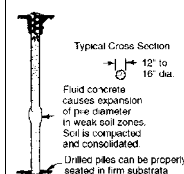
Pile type	Timber	Steel	Cast-in-place concrete piles (shells driven without mandrel)	Cast-in-place concrete piles (shells withdrawn)
Advantages	Comparatively low initial cost Permanently submerged piles are resistant to decay Easy to handle	Easy to splice High capacity Small displacement Able to penetrate through light obstructions	Can be redriven Shell not easily damaged	Initial economy
Remarks	Best suited for friction pile in granular material	Best suited for end bearing on rock Reduce allowable capacity for corrosive locations	Best suited for friction piles of medium length	Allowable load on pedestal pile is controlled by bearing capacity of stratum immediately below pile
Typical illustrations				

TABLE 6.15 Typical Pile Characteristics and Uses (*Continued*)

Pile type	Concrete filled steel pipe piles	Composite piles	Precast concrete (including prestressed)	Cast in place (thin shell driven with mandrels)	Auger placed pressure-injected concrete (grout) piles
Maximum length	Practically unlimited	55 m (180 ft)	30 m (100 ft) for precast 60 m (200 ft) for prestressed	30 m (100 ft) for straight sections 12 m (40 ft) for tapered sections	9–25 m (30–80 ft)
Optimum length	12–36 m (40–120 ft)	18–36 m (60–120 ft)	12–15 m (40–50 ft) for precast 18–30 m (60–100 ft) for prestressed	12–18 m (40–60 ft) for straight 5–12 m (16–40 ft) for tapered	12–18 m (40–60 ft)
Applicable material specifications	ASTM A36 for core ASTM A252 for pipe ACI Code 318 for concrete	ACI Code 318 for concrete ASTM A36 for structural section ASTM A252 for steel pipe ASTM D25 for timber	ASTM A15 reinforcing steel ASTM A82 cold-drawn wire ACI Code 318 for concrete	ACI	See ACI[a]
Recommended maximum stresses	0.40 f_y reinforcement $<$ 205 MPa (30 ksi) 0.50 f_y for core $<$ 175 MPa (25 ksi) 0.33 f'_c for concrete	Same as concrete in other piles Same as steel in other piles Same as timber piles for wood composite	0.33f'_c unless local building code is less; 0.4 f_y for reinforced unless prestressed	0.33 f'_c; f_s = 0.4 f_y if shell gauge is $\leq$ 14; use f_s = 0.35 f_y if shell thickness $\geq$ 3 mm	0.225–0.4f'_c
Maximum load for usual conditions	1800 kN (400 kips) without cores 18,000 kN (4000 kips) for large sections with steel cores	1800 kN (400 kips)	8500 kN (2000 kips) for prestressed 900 kN (200 kips) for precast	675 kN (150 kips)	700 kN (160 kips)
Optimum-load range	700–1100 kN (160–250 kips) without cores 4500–14,000 kN (1000–3100 kips) with cores	250–725 kN (60–160 kips)	350–3500 kN (80–800 kips)	250–550 kN (60–120 kips)	350–550 kN (80–120 kips)

TABLE 6.15 Typical Pile Characteristics and Uses (*Continued*)

Pile type	Concrete filled steel pipe piles	Composite piles	Precast concrete (including prestressed)	Cast in place (thin shell driven with mandrels)	Auger placed pressure-injected concrete (grout) piles
Disadvantages	High initial cost Displacement for closed-end pipe	Difficult to attain good joint between two materials	Difficult to handle unless prestressed High initial cost Considerable displacement Prestressed difficult to splice	Difficult to splice after concreting Redriving not recommended Thin shell vulnerable during driving Considerable displacement	Dependence on workmanship Not suitable in compressible soil
Advantages	Best control during installation No displacement for open-end installation Open-end pipe best against obstructions High load capacitites Easy to splice	Considerable length can be provided at comparatively low cost	High load capacities Corrosion resistance can be attained Hard driving possible	Initial economy Taped sections provide higher bearing resistance in granular stratum	Freedom from noise and vibration Economy High skin friction No splicing
Remarks	Provides high bending resistance where unsupported length is loaded laterally	The weakest of any material used shall govern allowable stresses and capacity	Cylinder piles in particular are suited for bending resistance	Best suited for medium-load friction piles in granular materials	Patented method
Typical illustrations					

[a] ACI Committee 543, "Recommendations for Design, Manufacture, and Installation of Concrete Piles," *JACI,* August 1973, October 1974.

Sources: NAVFAC DM-7.2, 1982 and J. E. Bowles, "Foundation Analysis and Design," 3d ed., McGraw-Hill Publishing, Co., New York.

Stresses given for steel piles and shells are for noncorrosive locations. For corrosive locations estimate possible reduction in steel cross section or provide protection from corrosion.

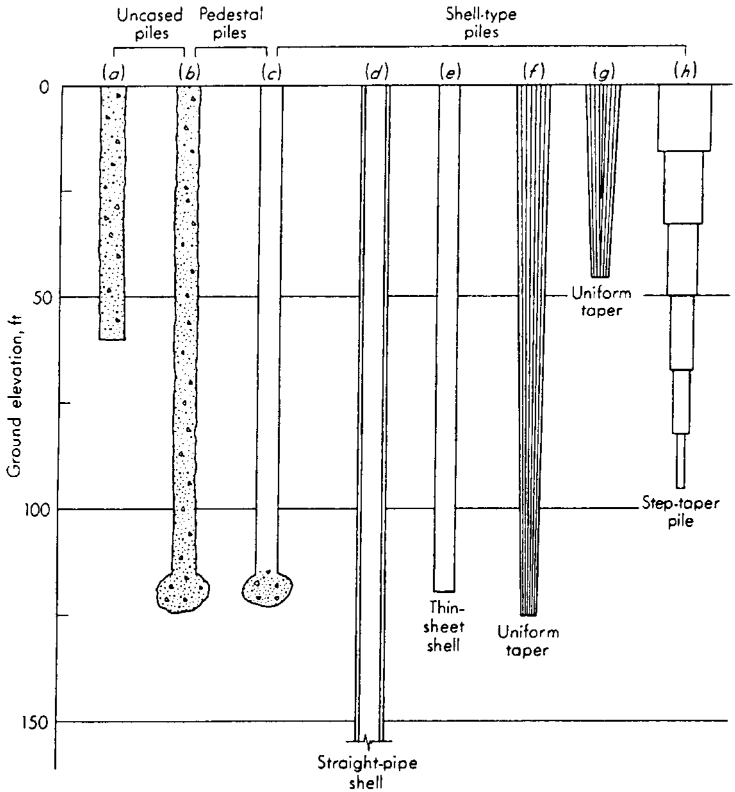


FIGURE 6.42 Common types of cast-in-place concrete piles: (*a*) Uncased pile; (*b*) Franki uncased-pedestal pile; (*c*) Franki cased-pedestal pile; (*d*) welded or seamless pipe pile; (*e*) cased pile using a thin sheet shell; (*f*) monotube pile; (*g*) uniform tapered pile; (*h*) step-tapered pile. (*From J. E. Bowles, "Foundation Analysis and Design," 3d ed., McGraw-Hill Publishing Co., New York, with permission of McGraw-Hill, Inc.*)

6.9 FOUNDATION EXCAVATIONS

There are many different types of excavations performed during the construction of a project. For example, soil may be excavated from the cut or borrow area and then used as fill (see Art. 6.10). Another example is the excavation of a shear key or buttress that will be used to stabilize a slope or landslide. Other examples of excavations are as follows:

1. **Footing Excavations.** This type of service involves measuring the dimension of geotechnical elements (such as the depth and width of footings) to make sure that they conform to the requirements of the construction plans. This service is often performed at the same time as the field observation to confirm bearing conditions.

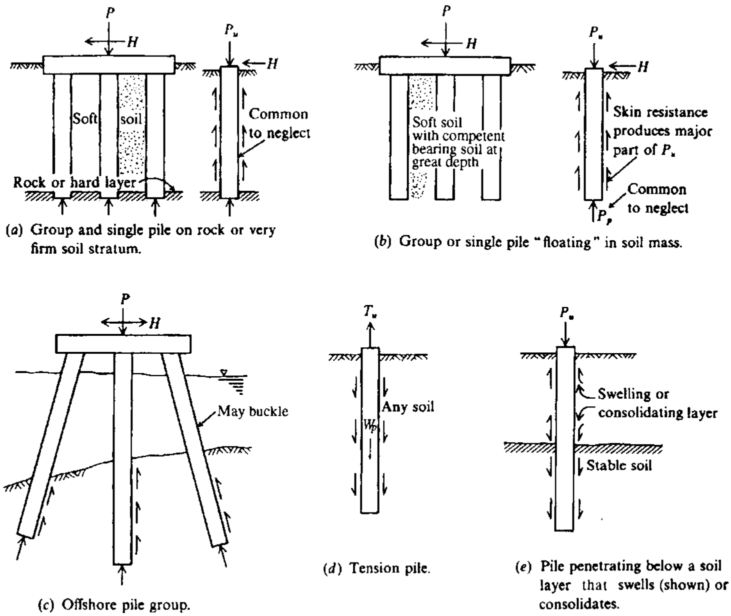


FIGURE 6.43 Typical pile configurations. (*From J. E. Bowles, "Foundation Analysis and Design," 3d ed., McGraw-Hill Publishing Co., New York, with permission of McGraw-Hill, Inc.*)

2. **Excavation of Piers.** As with the excavation of footings, the geotechnical engineer may be required to confirm embedment depths and bearing conditions for piers. Figure 6.44 presents typical steps in the construction of a drilled pier.
3. **Open Excavations.** An open excavation is defined as an excavation that has stable and unsupported side slopes. Table 6.16 presents a discussion of the general factors that control the excavation stability, and Table 6.17 lists factors that control the stability of excavation slopes in some problem soils.
4. **Braced Excavations.** A braced excavation is defined as an excavation where the sides are supported by retaining structures. Figure 6.45 shows common types of retaining systems and braced excavations. Table 6.18 lists the design considerations for braced excavations, and Table 6.19 indicates factors that are involved in the choice of a support system for a deep excavation.

6.10 GRADING AND OTHER SITE IMPROVEMENT METHODS

Since most building sites start out as raw land, the first step in site construction work usually involves the grading of the site. Grading is defined as any operation consisting of excavation, filling, or a combination thereof. A typical grading process could include some or all of the following:

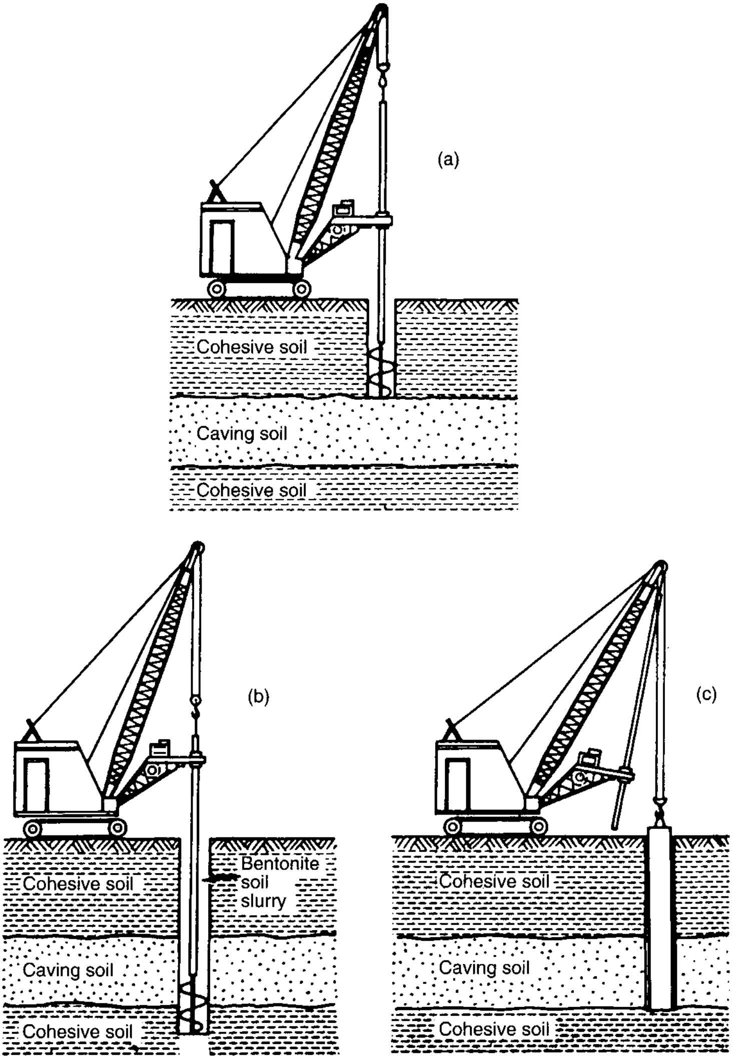


FIGURE 6.44 Typical steps in the construction of a drilled pier: (*a*) dry augering through self-supporting cohesive soil; (*b*) augering through water bearing cohesionless soil with aid of slurry; (*c*) setting the casing.

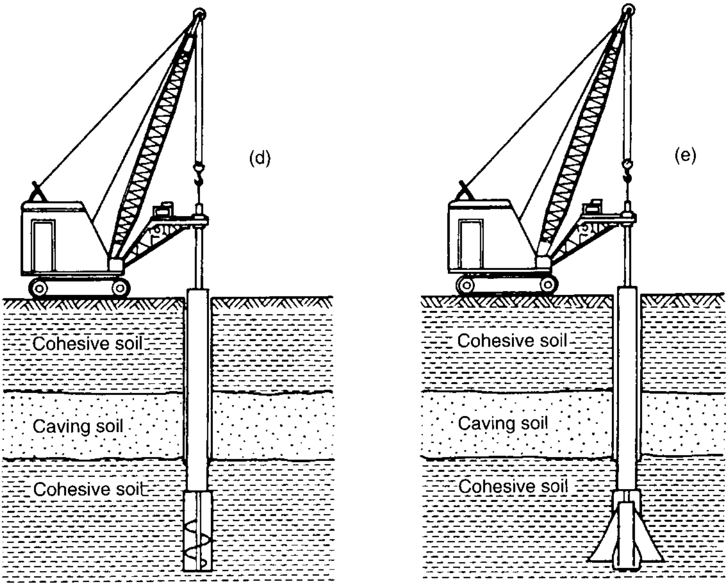


FIGURE 6.44 (*d*) dry augering into cohesive soil after sealing; (*e*) forming a bell. (*After O'Neill and Reese. Reproduced from R. B. Peck, W. E. Hanson, and T. H. Thornburn, "Foundation Engineering," John Wiley & Sons, Inc., New York.*) (*Continued*)

1. **Easements.** The first step in the grading operation is to determine the location of any on-site utilities and easements. The on-site utilities and easements often need protection so that they are not damaged during the grading operation.
2. **Clearing, Brushing, and Grubbing.** Clearing, brushing, and grubbing are defined as the removal of vegetation (grass, brush, trees, and similar plant types) by mechanical means. It is important that this debris be removed from the site and not accidentally placed within the structural fill mass.
3. **Cleanouts.** This grading process deals with the removal of unsuitable bearing material at the site, such as loose or porous alluvium, colluvium, peat, muck, and uncompacted fill.
4. **Benching (Hillside Areas).** Benching is defined as the excavation of relatively level steps into earth material on which fill is to be placed.
5. **Canyon Subdrain.** A subdrain is defined as a pipe and gravel or similar drainage system placed in the alignment of canyons or former drainage channels. After placement of the subdrain, structural fill is placed on top of the subdrain.
6. **Scarifying and Recompaction.** In flat areas that have not been benched, scarifying and recompaction of the ground surface is performed by compaction equipment in order to get a good bond between the in-place material and compacted fill.

TABLE 6.16 General Factors That Control the Stability of the Excavation Slopes

Construction activity	Objectives	Comments
Dewatering	In order to prevent boiling, softening, or heave of the excavation bottom, reduce lateral pressures on sheeting, reduce seepage pressures on face of open cut, and eliminate piping of fines through sheeting.	Investigate soil compressibility and effect of dewatering on settlement of nearby structures; consider recharging or slurry wall cutoff. Examine for presence of lower aquifer and need to dewater. Install piezometers if needed. Consider effects of dewatering in cavity-laden limestone. Dewater in advance of excavation.
Excavation and grading (also see Art. 6.10)	Utility trenches, basement excavations, and site grading.	Analyze safe slopes or bracing requirements, and effects of stress reduction on overconsolidated, soft, or swelling soils and shales. Consider horizontal and vertical movements in adjacent areas due to excavation and effect on nearby structures. Keep equipment and stockpiles a safe distance from the top of the excavation.
Excavation wall construction	To support vertical excavation walls, and to stabilize trenching in limited space.	See Art. 6.7 for retaining wall design. Reduce earth movements and bracing stresses, where necessary, by installing lagging on front flange of soldier pile. Consider effect of vibrations due to driving sheet piles or soldier piles. Consider dewatering requirements as well as wall stability in calculating sheeting depth. Movement monitoring may be warranted.
Blasting	To remove or to facilitate the removal of rock in the excavation.	Consider the effect of vibrations on settlement or damage to adjacent areas. Design and monitor or require the contractor to design and monitor blasting in critical areas, and require a pre-construction survey of nearby structures.
Anchor or strut installation	To obtain support system stiffness and interaction.	Major excavations require careful installation and monitoring, e.g., case anchor holes in collapsible soil, measure stress in ties and struts, etc.

Sources: NAVFAC DM-7.2, 1982, Clough and Davidson 1977, and Departments of the Army and the Air Force 1979. G. W. Clough and R. R. Davidson, ''Effects of Construction on Geotechnical Performance,'' and Department of the Army and the Air Force, ''Soils and Geology, Procedures for Foundation Design.''

TABLE 6.17 Stability of Excavation Slopes in Some Problem Soils

Topic	Discussion
General discussion	The depth and slope of an excavation and groundwater conditions control the overall stability and movements of open excavations. Factors that control the stability of the excavation for different material types are as follows: 1. Rock: For rock, stability is controlled by depths and slopes of excavation, particular joint patterns, in-situ stresses, and groundwater conditions. 2. Granular Soils: For granular soils, instability usually does not extend significantly below the bottom of the excavation provided that seepage forces are controlled. 3. Cohesive Soils: For cohesive soils, stability typically involves side slopes but may also include the materials well below the bottom of the excavation. Instability of the bottom of the excavation, often referred to as bottom heave, is affected by soil type and strength, depth of cut, side slope and/or berm geometry, groundwater conditions, and construction procedures.
Stiff-fissured clays and shales	Field shear resistance may be less than suggested by laboratory testing. Slope failures may occur progressively and shear strengths are reduced to the residual value compatible with relatively large deformations. Some case histories suggest that the long-term performance is controlled by the drained residual friction angle. The most reliable design would involve the use of local experience and recorded observations.
Loess and other collapsible soil	Such soils have a strong potential for collapse and erosion of relatively dry materials upon wetting. Slopes in loess are frequently more stable when cut vertical to prevent water infiltration. Benches at intervals can be used to reduce effective slope angles. Evaluate potential for collapse as described in Art. 6.5.5.
Residual soil	Depending on the weathering profile from the parent rock, residual soil can have a significant local variation in properties. Guidance based on recorded observations provides a prudent basis for design.
Sensitive clay	Very sensitive and quick clays have a considerable loss of strength upon remolding, which could be generated by natural or man-made disturbance. Minimize disturbance and use total stress analysis based on undrained shear strength from unconfined compression tests or field vane tests.
Talus	Talus is characterized by loose aggregation of rock that accumulates at the foot of rock cliffs. Stable slopes are commonly between 1.25:1 to 1.75:1 (horizontal:vertical). Instability is often associated with abundance of water, mostly when snow is melting.
Loose sands	Loose sands may settle under blasting vibrations, or liquefy, settle, and lose shear strength if saturated. Such soils are also prone to erosion and piping.
Engineering evaluation	Slope stability analyses may be used to evaluate the stability of open excavations in soils where the behavior of such soils can be reasonably determined by field investigations, laboratory testing, and engineering analysis. As described above, in certain geologic formations stability is controlled by construction procedures, side effects during and after excavation, and inherent geologic planes of weaknesses.

Sources: NAVFAC DM-7.2, 1982 and Clough and Davidson 1977. G. W. Clough and R. R. Davidson, ‘‘Effects of Construction on Geotechnical Performance,’’ and Department of the Army and the Air Force, ‘‘Soils and Geology, Procedures for Foundation Design.’’

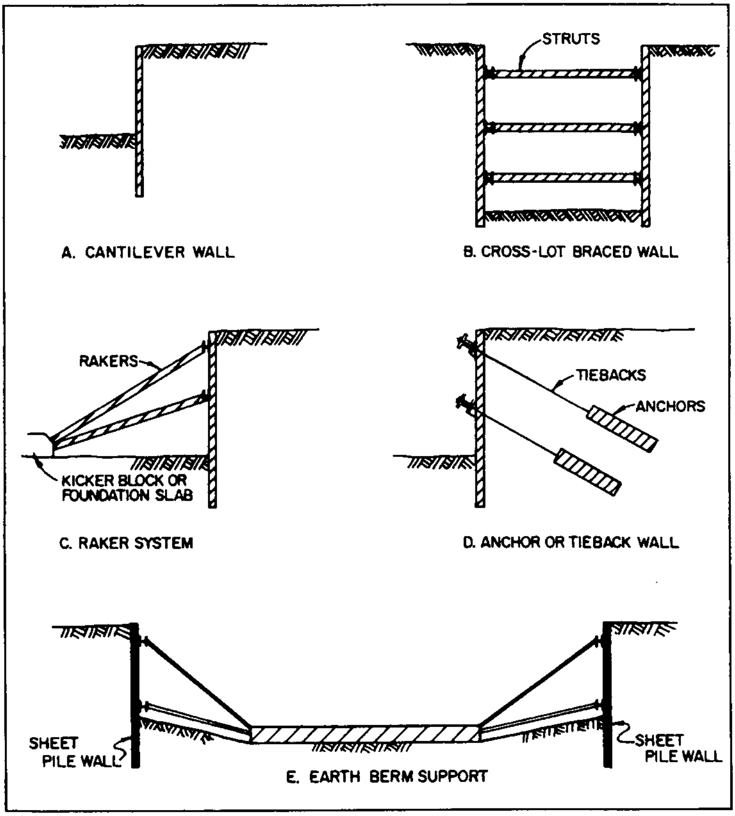


FIGURE 6.45 Common types of retaining systems and braced excavations. (*From NAVFAC DM-7.2, 1982.*)

7. **Cut and Fill Rough Grading Operations.** Rough grading operations involve the cutting of earth materials from high areas and compaction of fill in low areas, in conformance with grading plans. Other activities could be performed during rough grading operations, such as:

   (*a*) **Ripping or Blasting of Rock.** Large rock fragments can be removed from the site or disposed of in windrows.

   (*b*) **Cut-Fill Transition.** A cut-fill transition is the location in a building pad where on one side the pad has been cut down, exposing natural or rock material, while on the other side fill has been placed. One method to deal with a cut-fill transition is to over-excavate the cut portion of the pad and replace it with compacted fill.

   (*c*) **Slope Stabilization.** Examples of slope stabilization using earth materials include stabilization fill, buttress fill, drainage buttress, and shear keys. Such devices should be equipped with backdrain systems.

   (*d*) **Fill Slopes.** When creating a fill slope, it is often difficult to compact the outer edge of the fill mass. Because there is no confining pressure, the soil

TABLE 6.18 Design Considerations for Braced Excavations

Design factor	Comments
Water loads	Often greater than earth loads on an impervious wall. Recommend piezometers during construction to monitor water levels. Should also consider possible lower water pressures as a result of seepage of water through or under the wall. Dewatering can be used to reduce the water loads. Seepage under the wall reduces the passive resistance.
Stability	Consider the possible instability in any berm or exposed slope. The sliding potential beneath the wall or behind the tiebacks should also be evaluated. For weak soils, deep seated bearing failure due to the weight of the supported soil should be checked. Also include in stability analysis the weight of surcharge or weight of other facilities in close proximity to the excavation.
Piping	Piping due to a high groundwater table causes a loss of ground, especially for silty and fine sands. Difficulties occur due to flow of water beneath the wall, through bad joints in the wall, or through unsealed sheet pile handling holes. Dewatering may be required.
Movements	Movements can be minimized through the use of a stiff wall supported by preloaded tiebacks or a braced system.
Dewatering and recharge	Dewatering reduces the loads on the wall system and minimizes the possible loss of ground due to piping. Dewatering may cause settlements and in order to minimize settlements, there may be the need to recharge outside of the wall system.
Surcharge	Construction materials are usually stored near the wall systems. Allowances should always be made for surcharge loads on the wall system.
Prestressing of tieback anchors	In order to minimize soil and wall movements, it is useful to remove slack by prestressing tieback anchors.
Construction sequence	The amount of wall movement is dependent on the depth of the excavation. The amount of load on the tiebacks is dependent on the amount of wall movement which occurs before they are installed. Movements of the wall should be checked at every major construction stage. Upper struts should be installed as early as possible.
Temperature	Struts may be subjected to load fluctuations due to temperature differences. This may be important for long struts.
Frost penetration	In cold climates, frost penetration can cause significant loading on the wall system. Design of the upper portion of the wall system should be conservative. Anchors may have to be heated. Freezing temperatures also can cause blockage of flow of water and thus unexpected buildup of water pressure.
Earthquakes	Seismic loads may be induced during an earthquake.
Factors of safety	The following are suggested minimum factors of safety (F) for overall stability. Note that these values are suggested guidelines only. Design factors of safety depend on project requirements. Earth Berms: Permanent, F = 2.0 Temporary, F = 1.5 Cut Slopes: Permanent, F = 1.5 Temporary, F = 1.3 General Stability: Permanent, F = 1.5 Temporary, F = 1.3 Bottom Heave: Permanent, F = 2.0 Temporary, F = 1.5

Source: NAVFAC DM-7.2, 1982.

TABLE 6.19 Factors Involved in the Choice of a Support System for an Excavation

Requirements	Type of support system	Comments
Open excavation area	Tiebacks or rakers. For shallow excavation, use cantilever walls.	Consider design items listed in Table 6.18.
Low initial cost	Soldier pile or sheet pile walls. Consider combined soil slope and wall.	Consider design items listed in Table 6.18.
Use as part of permanent structure	Diaphragm or pier walls.	Diaphragm wall is the most common type of permanent wall.
Subsurface conditions of deep, soft clay	Struts or rakers that support a diaphragm or pier wall.	Tieback capacity not adequate in soft clays.
Subsurface conditions of dense, gravelly sands or clay	Soldier pile, diaphragm wall, or pier wall.	Sheet piles may lose interlock on hard driving.
Subsurface conditions of overconsolidated clays	Struts, long tiebacks, or combination of tiebacks and struts.	High in-situ lateral stresses are relieved in overconsolidated soil. Lateral movements may be large and extend deep into the soil.
Avoid dewatering	Use diaphragm walls or possibly sheet pile walls in soft subsoils.	Soldier pile wall is too pervious for this application.
Minimize lateral movements of wall	Use high preloads on stiff strutted or tieback walls.	Analyze the stability of the bottom of the excavation.
Wide excavation (greater than 65 ft wide)	Use tiebacks or rackers.	Tiebacks are preferable except in very soft clay soils.
Narrow excavation (less than 65 ft wide)	Use cross-excavation struts.	Struts are more economical, but tiebacks still may be preferred in order to keep the excavation open.

Note: Deep excavation is defined as an excavation that is more than 20 feet (6 m) below ground surface.
Source: NAVFAC DM-7.2, 1982.

deforms downslope without increasing in density. To deal with this situation, the slope can be overbuilt and then cut back to the compacted core. The second-best alternative is to use conventional construction procedures such as back-rolling techniques or by using a bulldozer to track-walk the slope.

(*e*) **Revision of Grading Operations.** Every grading job is different, and there could be a change in grading operations based on field conditions.

8. **Fine Grading** (also known as **Precise Grading).** At the completion of the rough grading operations, fine grading is performed to obtain the finish elevations in accordance with the precise grading plan.

9. **Slope Protection and Erosion Control.** Although this is usually not the responsibility of the grading contractor, upon completion of the fine grading, slope protection and permanent erosion control devices are installed.
10. **Trench Excavations.** Utility trenches are excavated in the proposed road alignments and building pads for the installation of the on-site utilities. The excavation and compaction of utility trenches is often part of the grading process. Once the utility lines are installed, scarifying and recompaction of the road subgrade is performed and base material is placed and compacted.
11. **Footing and Foundation Excavations.** Although this is usually not part of the grading operation, the footing and foundation elements are then excavated (see Art. 6.9).

6.10.1 Compaction Fundamentals

An important part of the grading of the site often includes the compaction of fill. Compaction is defined as the densification of a fill by mechanical means. This physical process of getting the soil into a dense state can increase the shear strength, decrease the compressibility, and decrease the permeability of the soil. There are four basic factors that affect compaction:

1. **Soil Type.** Nonplastic (i.e., granular) soil, such as sands and gravels, can be effectively compacted by using a vibrating or shaking type of compaction operation. Plastic (i.e., cohesive) soil, such as silts and clays, is more difficult to compact and requires a kneading or manipulation type of compaction operation. If the soil contains oversize particles, such as coarse gravel and cobbles, these particles tend to interfere with the compaction process and reduce the effectiveness of compaction for the finer soil particles. Typical values of dry density for different types of compacted soil are listed in Table 6.20.
2. **Material Gradation.** Those soils that have a well-graded grain size distribution can generally be compacted into a denser state than a poorly graded soil that is composed of soil particles of about the same size. For example, a well-graded decomposed granite (DG) can have a maximum dry density of 2.2 Mg/m^3 (137 pcf), while a poorly graded sand can have a maximum dry density of only 1.6 Mg/m^3 (100 pcf, Modified Proctor).
3. **Water Content.** The water content is an important parameter in the compaction of soil. Water tends to lubricate the soil particles thus helping them slide into dense arrangements. However, too much water and the soil becomes saturated and often difficult to compact. There is an optimum water content at which the soil can be compacted into its densest state for a given compaction energy. Typical optimum moisture contents (Modified Proctor) for different soil types are as follows:

   (***a***) **Clay of High Plasticity** (CH): optimum moisture content $\geq$ 18%
   (***b***) **Clay of Low Plasticity** (CL): optimum moisture content = 12 to 18%
   (***c***) **Well-Graded Sand** (SW): optimum moisture content = 10%
   (***d***) **Well-Graded Gravel** (GW): optimum moisture content = 7%

   Some soils may be relatively insensitive to compaction water content. For example, open-graded gravels and clean coarse sands are so permeable that water simply drains out of the soil or is forced out of the soil during the compaction process. These types of soil can often be placed in a dry state and then vibrated into dense particle arrangements.

TABLE 6.20 Characteristics of Compacted Subgrade for Roads and Airfields (from *The Unified Soil Classification System,* U.S. Army, 1960)

Major divisions (1)	Subdivisions (2)	USCS symbol (3)	Name (4)	Value as subgrade (no frost action) (5)	Potential frost action (6)
Coarse-grained soils	Gravel and gravelly soils	GW	Well-graded gravels or gravel-sand mixtures, little or no fines	Excellent	None to very slight
		GP	Poorly graded gravels or gravelly sands, little or no fines	Good to excellent	None to very slight
		GM	Silty gravels, gravel-sand-silt mixtures	Good to excellent	Slight to medium
		GC	Clayey gravels, gravel-sand-clay mixtures	Good	Slight to medium
	Sand and sandy soils	SW	Well-graded sands or gravelly sands, litttle or no fines	Good	None to very slight
		SP	Poorly graded sands or gravelly sands, little or no fines	Fair to good	None to very slight
		SM	Silty sands, sand-silt mixtures	Fair to good	Slight to high
		SC	Clayey sands, sand-clay mixtures	Poor to fair	Slight to high
Fine-grained soils	Silts and clays with liquid limit less than 50	ML	Inorganic silts, rock flour, silts of low plasticity	Poor to fair	Medium to very high
		CL	Inorganic clays of low plasticity, gravelly clays, sandy clays, etc.	Poor to fair	Medium to high
		OL	Organic silts and organic clays of low plasticity	Poor	Medium to high
	Silts and clays with liquid limit greater than 50	MH	Inorganic silts, micaceous silts, silts of high plasticity	Poor	Medium to very high
		CH	Inorganic clays of high plasticity, fat clays, silty clays, etc.	Poor to fair	Medium
		OH	Organic silts and organic clays of high plasticity	Poor to very poor	Medium
Peat	Highly organic	PT	Peat and other highly organic soils	Not suitable	Slight

TABLE 6.20 Characterics of Compacted Subgrade for Roads and Airfields (from *The Unified Soil Classification System,* U.S. Army, 1960) (*Continued*)

Compressibility (7)	Drainage properties (8)	Compaction equipment (9)	Typical dry densities (10)		CBR (11)	Sub. mod.,[a] pci (12)
			pcf	Mg/m^3		
Almost none	Excellent	Crawler-type tractor, rubber-tired roller, steel-wheeled roller	125–140	2.00–2.24	40–80	300–500
Almost none	Excellent	Crawler-type tractor, rubber-tired roller, steel-wheeled roller	110–140	1.76–2.24	30–60	300–500
Very slight to slight	Fair to very poor	Rubber-tired roller, sheepsfoot roller	115–145	1.84–2.32	20–60	200–500
Slight	Poor to very poor	Rubber-tired roller, sheepsfoot roller	130–145	2.08–2.32	20–40	200–500
Almost none	Excellent	Crawler-type tractor, rubber-tired roller	110–130	1.76–2.08	20–40	200–400
Almost none	Excellent	Crawler-type tractor, rubber-tired roller	105–135	1.68–2.16	10–40	150–400
Very slight to medium	Fair to poor	Rubber-tired roller, sheepsfoot roller	100–135	1.60–2.16	10–40	100–400
Slight to medium	Poor to very poor	Rubber-tired roller, sheepsfoot roller	100–135	1.60–2.16	5–20	100–300
Slight to medium	Fair to poor	Rubber-tired roller, sheepsfoot roller	90–130	1.44–2.08	15 or less	100–200
Medium	Practically impervious	Rubber-tired roller, sheepsfoot roller	90–130	1.44–2.08	15 or less	50–150
Medium to high	Poor	Rubber-tired roller, sheepsfoot roller	90–105	1.44–1.68	5 or less	50–100
High	Fair to poor	Sheepsfoot roller, rubber-tired roller	80–105	1.28–1.68	10 or less	50–100
High	Practically impervious	Sheepsfoot roller, rubber-tired roller	90–115	1.44–1.84	15 or less	50–150
High	Practically impervious	Sheepsfoot roller, rubber-tired roller	80–110	1.28–1.76	5 or less	25–100
Very high	Fair to poor	Compaction not practical	—	—	—	—

Source: U.S. Army, ''The Unified Soil Classification System.''
[a] Subgrade Modulus.

4. **Compaction Effort (or Energy).** The compactive effort is a measure of the mechanical energy applied to the soil. Usually, the greater the amount of compaction energy applied to a soil, the denser the soil will become. There are exceptions, such as pumping soils (i.e., saturated clays), which can not be densified by an increased compaction effort. Compactors are designed to use one or a combination of the following types of compaction effort:
   (***a***) Static weight or pressure
   (***b***) Kneading action or manipulation
   (***c***) Impact or a sharp blow
   (***d***) Vibration or shaking

The laboratory compaction test consists of compacting a soil at a known water content into a mold of specific dimensions using a certain compaction energy. The procedure is repeated for various water contents to establish the compaction curve. The most common testing procedures (compaction energy, number of soil layers in the mold, etc.) are the Modified Proctor (ASTM D 1557-91, 1998) and the Standard Proctor (ASTM D 698-91, 1998). The term **Proctor** is in honor of R. R. Proctor, who in 1933 showed that the dry density of a soil for a given compactive effort depends on the amount of water the soil contains during compaction.

For the Modified Proctor (ASTM D 1557-91, 1998, procedure A), the soil is compacted into a 10.2-cm (4-in) diameter mold that has a volume of 944 cm^3 (1/30 ft^3), where five layers of soil are compacted into the mold, with each layer receiving 25 blows from a 44.5-N (10-lbf) hammer that has a 0.46-m (18-in) drop. The Modified Proctor has a compaction energy of 2700 kN-m/m^3 (56,000 ft-lbf/ft^3). The test procedure is to prepare soil at a certain water content, compact the soil into the mold, and then, by recording the mass of soil within the mold, obtain the wet density of the compacted soil. By measuring the water content of the compacted soil, the dry density can be calculated. This compaction procedure is repeated for the soil at different water contents and then the data are plotted on a graph in order to obtain the compaction curve.

Figure 6.46 shows the compaction curves for various soils using the Modified Proctor compaction test. The compaction curves show the relationship between the dry density (or dry unit weight) and water content for a given compaction effort. The compaction data presented in Fig. 6.46 were obtained using the Modified Proctor specifications. The lines to the right of the compaction curves are each known as a **zero air voids curve.** These curves represent a condition of saturation (S = 100%) for a specified specific gravity. Note how the right side of the compaction curves are approximately parallel to the zero air voids curve. This is often the case for many soil types and can be used as a check on the laboratory test results.

The peak point of the compaction curve is the laboratory maximum dry density (or the maximum dry unit weight). The water content corresponding to the laboratory maximum dry density is known as the **optimum moisture content.** These laboratory data are important because it tells the grading contractor the best water content for the most efficient compaction of the soil.

The most common method of assessing the quality of the field compaction is to calculate the relative compaction (RC) of the fill, defined as: RC $= 100\ \rho_d/\rho_{d\max}$, where $\rho_{d\max}$ = laboratory maximum dry density and ρ_d = field dry density. The maximum dry density ($\rho_{d\max}$) is the peak point of the laboratory compaction curve. In order for ρ_d to be determined, a field density test must be performed. Field density tests can be classified as either destructive or nondestructive tests. Probably the most common destructive method of determining the field dry density is through

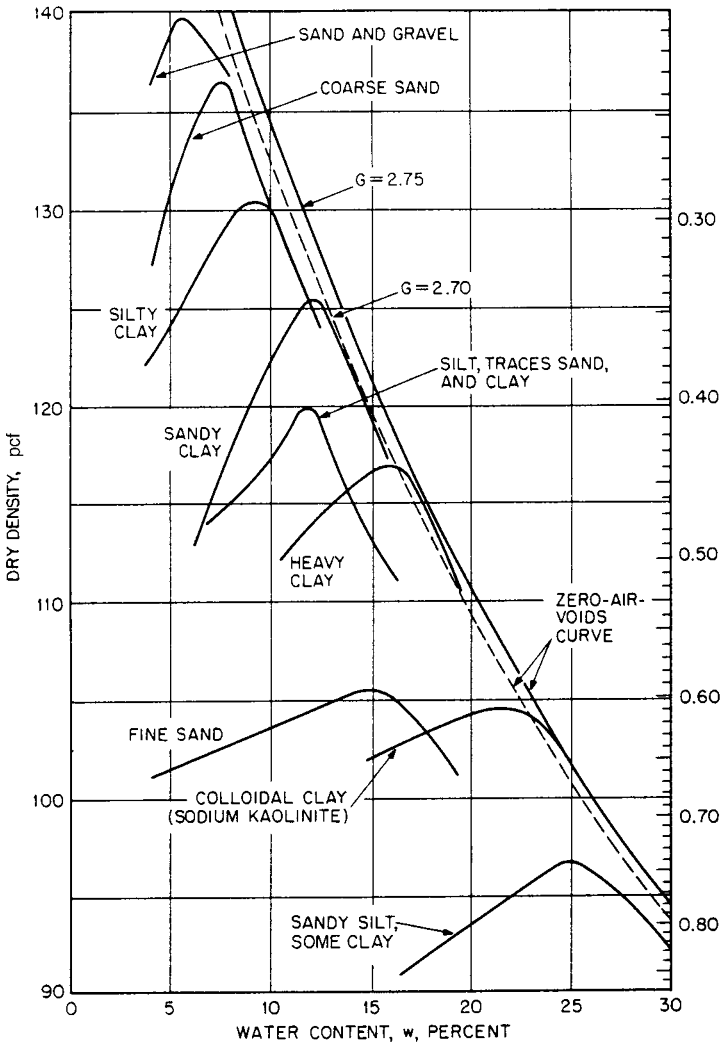


FIGURE 6.46 Compaction curves for various soils using the Modified Proctor laboratory test specifications.

the use of the sand cone apparatus. The test procedure consists of excavating a hole in the ground, filling the hole with sand using the sand cone apparatus, and then determining the volume of the hole based on the amount of sand required to fill the hole. Knowing the wet mass of soil removed from the hole divided by the volume of the hole enables the wet density of the soil to be calculated. The water content (w) of the soil extracted from the hole can be determined and thus the dry density (ρ_d) can then be calculated.

Another type of destructive test for determining the field dry density is the **drive cylinder.** This method involves the driving of a steel cylinder of known volume into the soil. Based on the mass of soil within the cylinder, the wet density can be calculated. Once the water content (w) of the soil is obtained, the dry density (ρ_d) of the fill can be calculated.

Probably the most common type of nondestructive field test is the **nuclear method.** In this method, the wet density is determined by the attenuation of gamma radiation. The nuclear method can give inaccurate results (density too high) where oversize particles are present, such as coarse gravel and cobbles. Likewise, if there is a large void in the source-detector path, then unusually low density values may be recorded.

6.10.2 Site Improvement Methods

If the expected settlement for a proposed structure is too large, then different foundation support or soil stabilization options must be evaluated. As discussed in Art. 6.8.2, one alternative is a deep foundation system that can transfer structural loads to adequate bearing material in order to bypass a compressible soil layer. Another option is to construct a floating foundation, which is a special type of deep foundation where the weight of the structure is balanced by the removal of soil and construction of an underground basement. Other alternatives include site improvement methods, such as the following (see Table 6.21):

Soil Replacement. As indicated in Table 6.21, there are basically two types of soil replacement methods: (1) removal and replacement, and (2) displacement. The first is the most common approach and consists of the removal of the compressible soil layer and replacement with structural fill during the grading operations. Usually the remove and replace grading option is economical only if the compressible soil layer is near the ground surface and the groundwater table is below the compressible soil layer or the groundwater table can be economically lowered.

Water Removal. Table 6.21 lists several different types of water removal site improvement techniques. If the site contains an underlying compressible cohesive soil layer, the site can be surcharged with a fill layer placed at ground surface. Vertical drains (such as wick drains or sand drains) can be installed in the compressible soil layer to reduce the drainage path and speed up the consolidation process. Once the compressible cohesive soil layer has had sufficient consolidation, the fill surcharge layer is removed and the building is constructed.

Site Strengthening. Many different methods can be used to strengthen the on-site soil (see Table 6.21). For example, deep vibratory techniques are often used to increase the density of loose sand deposits.

Grouting. In order to stabilize the ground, fluid grout can be injected into the ground to fill in joints, fractures, or underground voids. For the releveling of existing structures, one option is mudjacking, which has been defined as a process whereby a water and soil-cement or soil-lime cement grout is pumped beneath the slab, under pressure, to produce a lifting force that literally floats the slab to the desired position. Another commonly used site improvement technique is compaction grouting, which consists of intruding a mass of very thick-consistency grout into the soil, which both displaces and compacts the loose soil. Compaction grouting has proved successful in increasing the density of poorly compacted fill, alluvium, and compressible or collapsible soil. The advantages of compaction grouting are less expense and disturbance to the structure

TABLE 6.21 Site Improvement Methods

Method	Technique	Principles	Suitable soils	Remarks
Soil replacement methods	Remove and replace	Excavate weak or undesirable material and replace with better soils	Any	Limited depth and area where cost-effective; generally ≤ 30 ft
	Displacement	Overload weak soils so that they shear and are displaced by stronger fill	Very soft	Problems with mud-waves and trapped compressible soil under the embankment; highly dependent on specific site
Water removal methods	Trenching	Allows water drainage	Soft, fine-grained soils and hydraulic fills	Effective depth up to 10 ft; speed dependent on soil and trench spacing; resulting desiccated crust can improve site mobility
	Precompression	Loads applied prior to construction to allow soil consolidation	Normally consolidated fine-grained soil, organic soil, fills	Generally economical; long time may be needed to obtain consolidation; effective depth only limited by ability to achieve needed stresses
	Precompression with vertical drains	Shortens drainage path to speed consolidation	Same as above	More costly; effective depth usually limited to ≤100 ft
	Electro-osmosis	Electric current causes water to flow to cathode	Normally consolidated silts and silty clay	Expensive; relatively fast; usable in confined area; not usable in conductive soils; best for small areas

TABLE 6.21 Site Improvement Methods (*Continued*)

Method	Technique	Principles	Suitable soils	Remarks
Site strengthening methods	Dynamic compaction	Large impact loads applied by repeated dropping of a 5- to 35-ton weight; larger weights have been used	Cohesionless best; possible use for soils with fines; cohesive soils below groundwater table give poorest results	Simple and rapid; usable above and below the groundwater table; effective depths up to 60 ft; moderate cost; potential vibration damage to adjacent structures
	Vibro-compaction	Vibrating equipment densifies soils	Cohesionless soils with <20 percent fines	Can be efffective up to 100 feet depth; can achieve good density and uniformity; grid spacing of holes critical; relatively expensive
	Vibro-replacement	Jetting and vibration used to penetrate and remove soil; compacted granular fill then placed in hole to form support columns surrounded by undisturbed soil	Soft cohesive soils (s_u = 15 to 50 kPa, 300 to 1000 psf)	Relatively expensive
	Vibro-displacement	Similar to vibro-replacement except soil is displaced laterally rather than removed from the hole	Stiffer cohesive soils (s_u = 30 to 60 kPa, 600 to 1200 psf)	Relatively expensive
Grouting	Injection of grout	Fill soil voids with cementing agents to strengthen and reduce permeability	Wide spectrum of coarse- and fine-grained soils	Expensive; more expensive grouts needed for finer-grained soils; may use pressure injection, soil fracturing, or compaction techniques
	Deep mixing	Jetting or augers used to physically mix stabilizer and soil	Wide spectrum of coarse- and fine-grained soils	Jetting poor for highly cohesive clays and some gravelly soils; deep mixing best for soft soils up to 165 ft deep

TABLE 6.21 Site Improvement Methods (Continued)

Method	Technique	Principles	Suitable soils	Remarks
Thermal	Heat	Heat used to achieve irreversible strength gain and reduced water susceptibility	Cohesive soils	High energy requirements; cost limits practicality
	Freezing	Moisture in soil frozen to hold particles together and increase shear strength and reduce permeability	All soils below the groundwater table; cohesive soils above the groundwater table	Expensive; highly effective for excavations and tunneling; high groundwater flows troublesome; slow process
Geosynthetics	Geogrids, geotextiles, geonets, and geomembranes	Use geosynthetic materials for filters, erosion control, water barriers, drains, or soil reinforcing (see Art. 6.11)	Effective filters for all soils; reinforcement often used for soft soils	Widely used to accomplish a variety of tasks; commonly used in conjunction with other methods (e.g., strip drain with surcharge or to build a construction platform for site access)

Source: M. P. Rollings and R. S. Rollings, ''Geotechnical Materials in Construction,'' McGraw-Hill Publishing Co., New York.

than foundation underpinning, and it can be used to relevel the structure. The disadvantages are that analyzing the results is difficult, it is usually ineffective near slopes or for near-surface soils because of the lack of confining pressure, and the danger exists of filling underground pipes with grout.

Thermal. As indicated in Table 6.21, the thermal site improvement method consists of either heating or freezing the soil in order to improve its shear strength and reduce its permeability.

Figure 6.47 presents a summary of site-improvement methods as a function of soil grain size.

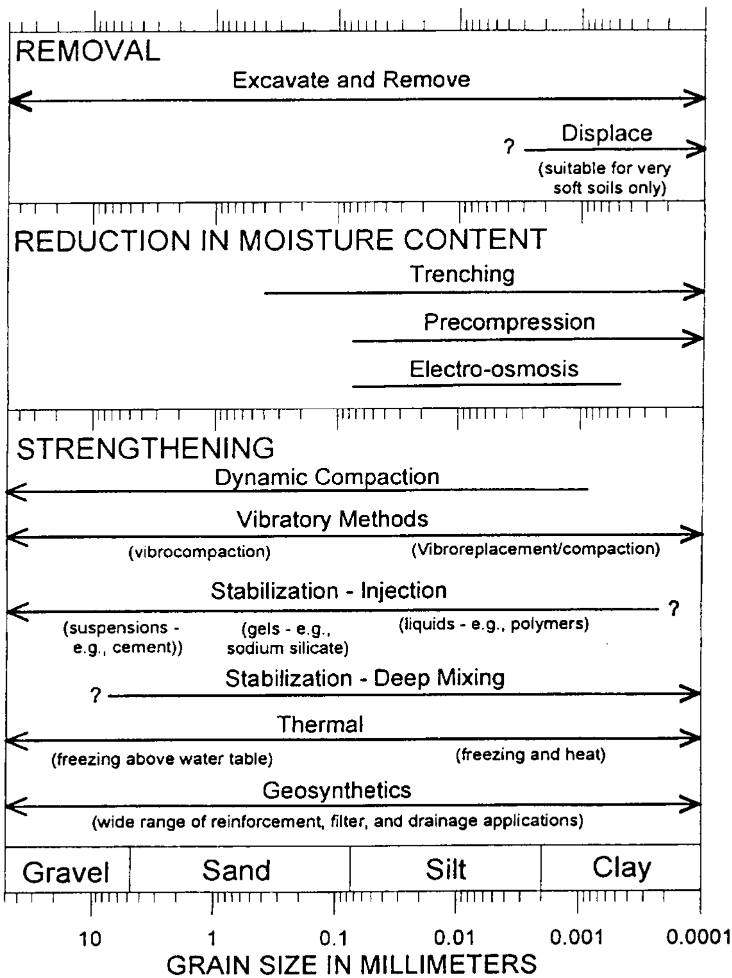


FIGURE 6.47 Site improvement methods as a function of soil grain size. (*Reproduced from M. P. Rollings and R. S. Rollings, "Geotechnical Materials in Construction," McGraw-Hill Publishing Co., New York, with permission of McGraw-Hill, Inc.*)

6.11 GEOSYNTHETICS

A **geosynthetic** is defined as a planar product manufactured from polymeric material and typically placed in soil to form an integral part of a drainage, reinforcement, or stabilization system. Common types of geosynthetics used during construction are as follows.

6.11.1 Geogrids

Figure 6.48 shows a photograph of a geogrid, which contains relatively high-strength polymer grids consisting of longitudinal and transverse ribs connected at their intersections. Geogrids have a large and open structure and the openings (i.e., apertures) are usually 0.5 to 4 in (1.3 to 10 cm) in length and/or width. Geogrids can be either biaxial or uniaxial, depending on the size of the apertures and shape of the interconnecting ribs. Geogrids are principally used as soil reinforcement, such as for subgrade stabilization, slope reinforcement, erosion control, mechanically stabilized earth retaining walls, and to strengthen the junction between the top of soft clays and overlying embankments. Geogrids are also used as an overlay in the construction or repair of asphalt pavements because they tend to reduce reflective cracking of the pavements.

Compacted soil tends to be strong in compression but weak in tension. The geogrid is just the opposite, strong in tension but weak in compression. Thus, layers of compacted soil and geogrid tend to complement each other and produce a soil mass having both high compressive and tensile strength. The open structure of the geogrid (see Fig. 6.48) allows the compacted soil to bond in the open geogrid spaces. Geogrids provide soil reinforcement by transferring local tensile stresses in the soil to the geogrid. Because geogrids are continuous, they also tend to transfer

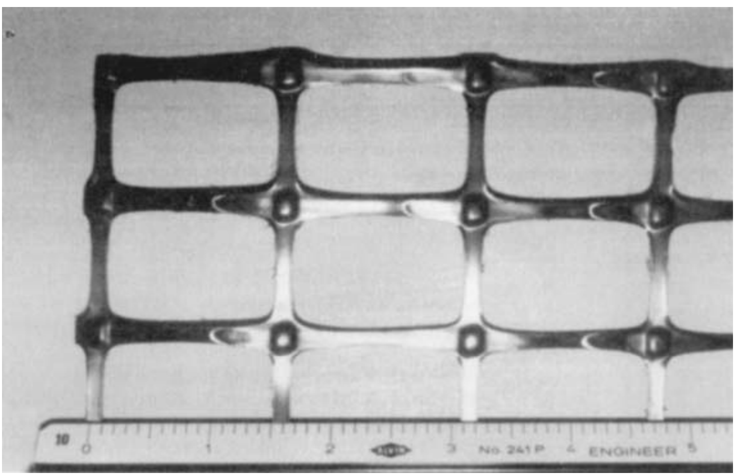


FIGURE 6.48 Photograph of a geogrid. (*Reproduced from M. P. Rollings and R. S. Rollings, "Geotechnical Materials in Construction," McGraw-Hill Publishing Co., New York, with permission of McGraw-Hill, Inc.*)

and redistribute stresses away from areas of high stress concentrations (such as beneath a wheel load).

Some of the limitations of geogrid are as follows:

1. **Ultraviolet Light.** Even geogrids produced of carbon black (i.e., ultraviolet stabilized geogrids) can degrade when exposed to long-term ultraviolet light. It is important to protect the geogrid from sunlight and cover the geogrid with fill as soon as possible.
2. **Non-uniform Tensile Strength.** Geogrids often have different tensile strengths in different directions as a result of the manufacturing process. For example, a Tensar SS-2 (BX1200) biaxial geogrid has an ultimate tensile strength of 2100 lb/ft in the main direction and only 1170 lb/ft in the minor (perpendicular) direction. It is essential that the engineer always check the manufacturer's specifications and determine the tensile strengths in the main and minor directions.
3. **Creep.** Polymer material can be susceptible to creep. Thus, it is important to use an allowable tensile strength that does allow for creep of the geosynthetic. Oftentimes, this allowable tensile design strength is much less than the ultimate strength of the geogrid. For example, for a Tensar SS-2 (BX1200) biaxial geogrid, the manufacturer's recommended tensile strength is about 300 lb/ft, which is only one-seventh the ultimate tensile strength (2100 lb/ft). The engineer should never apply an arbitrary factor of safety to the ultimate tensile strength, but rather obtain the allowable geogrid tensile design strength from the manufacturer.

6.11.2 Geotextiles

Geotextiles are the most widely used type of geosynthetic. Geotextiles are often referred to as **fabric**. For example, common construction terminology for geotextiles includes **geofabric, filter fabric, construction fabric, synthetic fabric,** and **road-reinforcing fabric.** As shown in Figs. 6.49 and 6.50, geotextiles are usually categorized as either woven or nonwoven, depending on the type of manufacturing process. Geotextiles are used for many different purposes, as follows:

1. **Soil Reinforcement.** Used for subgrade stabilization, slope reinforcement, and mechanically stabilized earth retaining walls. Also used to strengthen the junction between the top of soft clays and overlying embankments.
2. **Sediment Control.** Used as silt fences to trap sediment on-site.
3. **Erosion Control.** Installed along channels, under riprap, and used for shore and beach protection.
4. **Asphalt Overlay.** Used in asphalt overlays to reduce reflective cracking.
5. **Separation.** Used between two dissimilar materials, such as an open graded base and a clay subgrade, in order to prevent contamination.
6. **Filtration and Drainage.** Used in place of a graded filter where the flow of water occurs across (perpendicular to) the plane of the geotextile. For drainage applications, the water flows within the geotextile.

Probably the most common usage of geotextiles is for filtration (flow of water through the geotextile). For filtration, the geotextile should be at least 10 times more permeable than the soil. In addition, the geotextile must always be placed

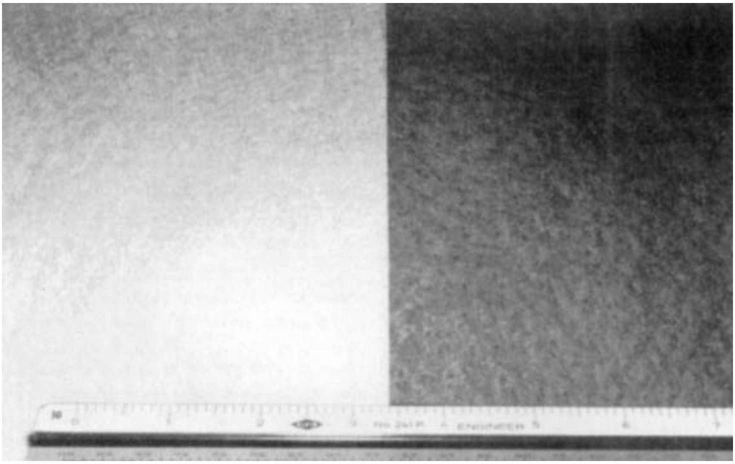


FIGURE 6.49 Photograph of nonwoven geotextiles. The geotextile on the left has no ultraviolet protection, while the geotextile on the right has ultraviolet protection. (*Reproduced from M. P. Rollings and R. S. Rollings, "Geotechnical Materials in Construction," McGraw-Hill Publishing Co., New York, with permission of McGraw-Hill, Inc.*)

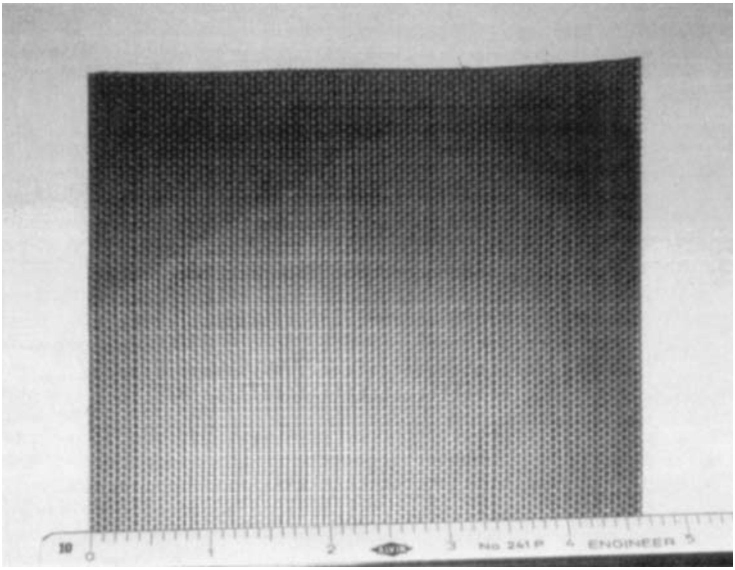


FIGURE 6.50 Photograph of a woven geotextile. (*Reproduced from M. P. Rollings and R. S. Rollings, "Geotechnical Materials in Construction," McGraw-Hill Publishing Co., New York, with permission of McGraw-Hill, Inc.*)

between a less permeable (i.e., the soil) and a more permeable (i.e., the open graded gravel) material. An inappropriate use of a geotextile would be to wrap it around a drainage pipe and then cover the geotextile with open-graded gravel. This is because the geotextile would then have more permeable material on both sides of the geotextile and it would tend to restrict flow.

Two important design properties for geotextiles used as filtration devices are that they have an adequate flow capacity and a proper soil retention capability:

1. **Flow Capacity.** Although specifications have been developed that limit the open area of the filtration geotextile to 10% or even 5%, it is best to have a larger open area to develop an adequate flow capacity.
2. **Soil Retention Capability.** The **apparent opening size** (AOS), also known as the **equivalent opening size** (EOS), determines the soil retention capability. The AOS is often expressed in terms of opening size (mm) or equivalent sieve size (e.g., AOS = 40–70 indicates openings equivalent to the No. 40 to No. 70 sieves). Obviously, if the geotextile openings are larger than the largest soil particle diameter, then all of the soil particles will migrate through the geotextile and clog the drainage system. A common recommendation is that the required AOS be less than or equal to D_{85} (grain size corresponding to 85% percent passing).

Some of the limitations of geotextile are as follows:

1. **Ultraviolet Light.** Geotextile that has no ultraviolet light protection can rapidly deteriorate. For example, certain polypropylene geotextiles lost 100% of their strength after only 8 weeks of exposure.
2. **Sealing of Geotextile.** When the geotextile is used for filtration, an impermeable soil layer can develop adjacent the geotextile if it has too low an open area or too small an AOS.
3. **Construction Problems.** Some of the more common problems related to construction with geotextiles are as follows (G. N. Richardson and D. C. Wyant, ‘‘Geotextiles Construction Criteria’’):
   (***a***) Fill placement or compaction techniques damage the geotextile.
   (***b***) Installation loads are greater than design loads, leading to failure during construction.
   (***c***) Construction environment leads to a significant reduction in assumed fabric properties, causing failure of the completed project.
   (***d***) Field seaming or overlap of the geotextile fails to fully develop desired fabric mechanical properties.
   (***e***) Instabilities during various construction phases may render a design inadequate even though the final product would have been stable.

6.11.3 Geomembranes

Common construction terminology for geomembranes includes **liners, membranes, visqueen, plastic sheets, and impermeable sheets.** Geomembranes are used almost exclusively as barriers to reduce water or vapor migration through soil (see Fig. 6.51). For example, a common usage for geomembranes is for the lining and capping systems in municipal landfills. For liners in municipal landfills, the thickness

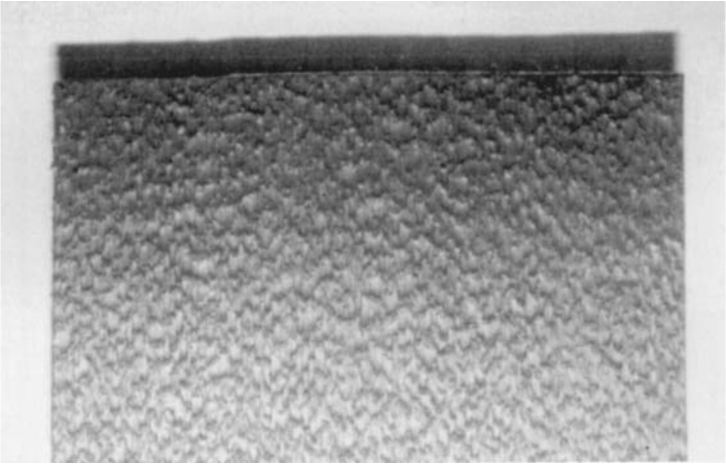


FIGURE 6.51 Photograph of a geomembrane, which has a surface texture for added friction. (*Reproduced from M. P. Rollings and R. S. Rollings, "Geotechnical Materials in Construction," McGraw-Hill Publishing Co., New York, with permission of McGraw-Hill, Inc.*)

of the geomembrane is usually at least 80 mil. In the United States, one mil is one-thousandth of an inch.

Some of the limitations of geomembranes are as follows:

1. **Puncture Resistance.** The geomembrane must be thick enough so that it is not punctured during installation and subsequent usage.
2. **Slide Resistance.** Slope failures have developed in municipal liners because of the smooth and low frictional resistance between the geomembrane and overlying or underlying soil. Textured geomembranes (such as shown in Fig. 6.51) have been developed to increase the frictional resistance of the geomembrane surface.
3. **Sealing of Seams.** A common cause of leakage through geomembranes is due to inadequate sealing of seams. The following are different methods commonly used to seal geomembrane seams (M. P. Rollings and R. S. Rollings, *"Geotechnical Materials in Construction,"* McGraw-Hill Publishing Co., New York):
   (***a***) **Thermal Fusion.** Suitable for thermoplastics. Adjacent surfaces are melted and then pressed together. Commercial equipment is available that uses a heated wedge (most common) or hot air to melt the materials. Also, ultrasonic energy can be used for melting rather than heat.
   (***b***) **Solvent-Based Systems.** Suitable for materials that are compatible with the solvent. A solvent is used with pressure to join adjacent surfaces. Heating may be used to accelerate the curing. The solvent may contain some of the geomembrane polymer already dissolved in the solvent liquid (bodied solvent) or an adhesive to improve the seam quality.
   (***c***) **Contact Adhesive.** Primarily suitable for thermosets. Solution is brushed onto surfaces to be joined, and pressure is applied to ensure good contact. Upon curing, the adhesive bonds the surfaces together.

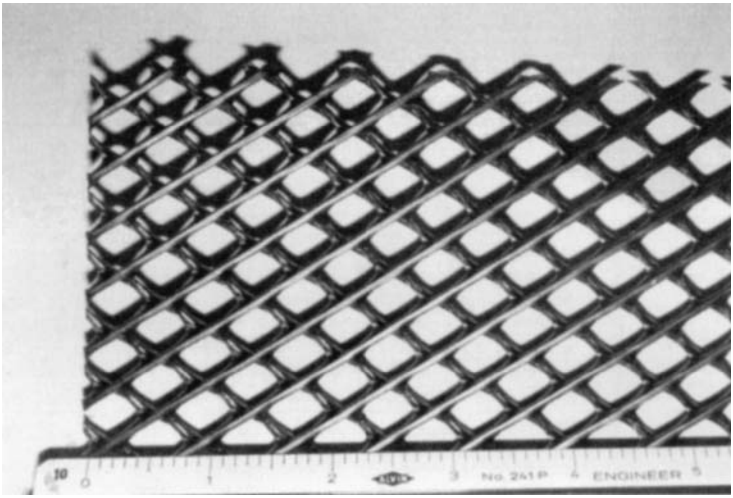


FIGURE 6.52 Photograph of a geonet. (*Reproduced from M. P. Rollings and R. S. Rollings, "Geotechnical Materials in Construction," McGraw-Hill Publishing Co., New York, with permission of McGraw-Hill, Inc.*)

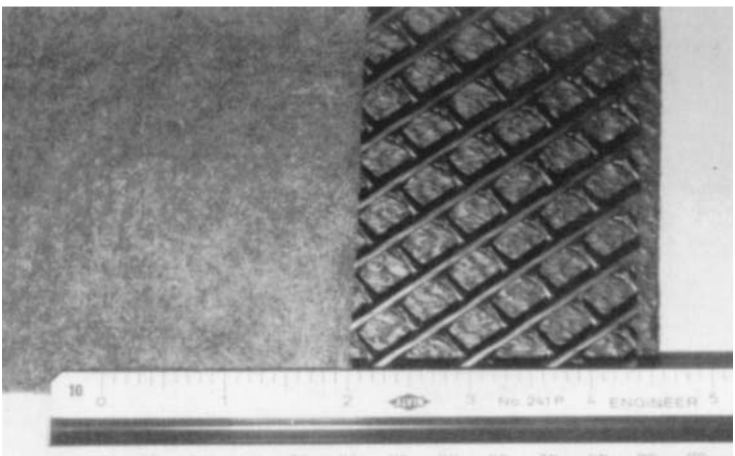


FIGURE 6.53 Photograph of a geocomposite. The geocomposite consists of a geonet having a textured geomembrane on top, and a filter fabric (geotextile) on the bottom. (*Reproduced from M. P. Rollings and R. S. Rollings, "Geotechnical Materials in Construction," McGraw-Hill Publishing Co., New York, with permission of McGraw-Hill, Inc.*)

(*d*) **Extrusion Welding.** Suitable for all polyethylenes. A ribbon of molten polymer is extruded over the edge (fillet weld) or between the geomembrane sheets (flat weld). This melts the adjacent surfaces, which are then fused together upon cooling.

6.11.4 Geonets and Geocomposites

Geonets are three-dimensional netlike polymeric materials used for drainage (flow of water within the geosynthetic). Figure 6.52 shows a photograph of a geonet. Geonets are usually used in conjunction with a geotextile and/or geomembrane and hence are technically a geocomposite.

Depending on the particular project requirements, different types of geosynthetics can be combined together to form a geocomposite. For example, a geocomposite consisting of a geotextile and a geomembrane provides for a barrier that has increased tensile strength and resistance to punching and tearing. Figure 6.53 shows a photograph of a geocomposite consisting of a textured geomembrane, geonet, and geotextile (filter fabric).

6.11.5 Geosynthetic Clay Liners

Geosynthetic clay liners are frequently used as liners for muncipal landfills. The geosynthetic clay liner typically consists of dry bentonite sandwiched between two geosynthetics. When moisture infiltrates the geosynthetic clay liner, the bentonite swells and creates a soil layer having a very low hydraulic conductivity, transforming it into an effective barrier to moisture migration.