8
Excavation and Trenching

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8.1
Introduction

The central focus of this chapter is occupational safety with respect to the man-made activity of excavation, defined by the Occupational Safety and Health Administration (OSHA) as "any man-made cut, cavity, trench, or depression in an earth surface, formed by earth removal." However, readers of this material are urged to recognize that excavation safety does not begin and end strictly with human activity.

It must be noted the removal of earth is more than a man-made activity like it is with trenching. Man is not the only force that "cuts" into the earth. Mother Nature "cuts" into the earth to create natural slopes such as hillsides, cliffs, sink holes, and river banks. The point is that although these types of natural slopes are not fully discussed here, it cannot be over-emphasized that Nature greatly impacts the stability and resulting safety of man-made slopes and excavations.

Man-made slopes commonly include embankments for highways or railroads, earth dams, river levees, dikes, landscaping swales, canals, and waterways. Excavations, in contrast, include such things as trenches for utility work, digging for foundations and footings, and excavating pits for basements or underground storage tanks.

Excavation hazards generally flow from at least three sources of risk: (i) the properties of the material being excavated, (ii) environmental impacts acting upon the excavated material, and (iii) hazards that are incidental to the nature of the work that may or may not come from others (i.e., the general public, contractors, owners, poorly trained fellow co-workers, material men, and suppliers). In this introduction, each of these hazards is briefly outlined (see Figures 8.1 and 8.2).

Soil is generally composed of solids, liquids, air voids, and at times organic matter. Soil weighs close to 3000 lb yd^{-3} (1780 kg m^{-3}). The complicated interaction between these distinct materials can cause soil to become unstable when excavated.

Risks associated with the failure of excavated material often include suffocation and crushing.

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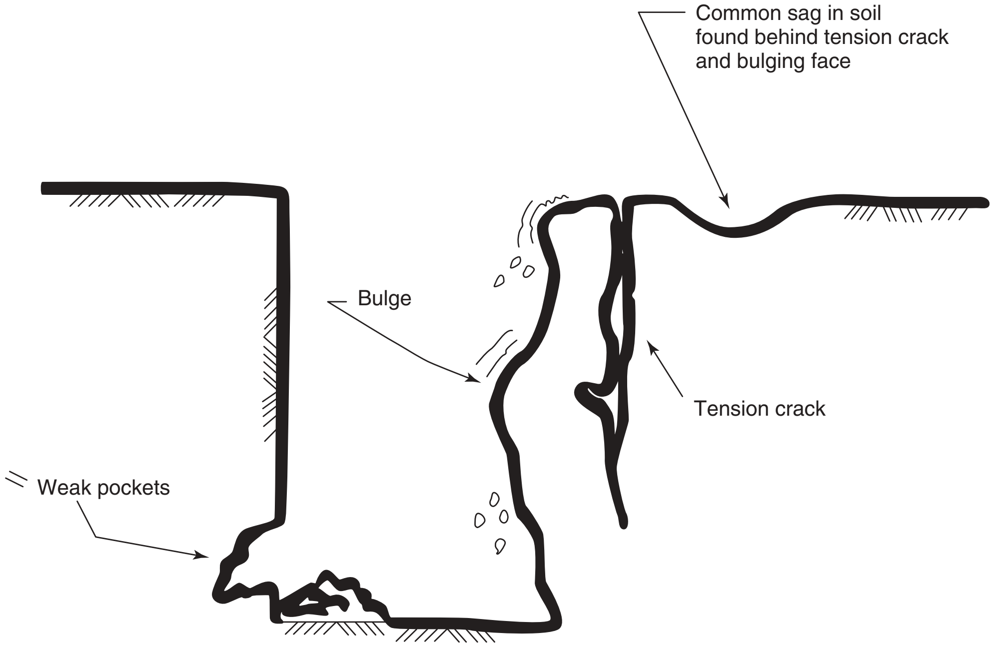


Figure 8.1 Typical modes of failure for excavated materials.

A ‘‘competent person’’ as defined by OSHA must understand the inherent risks due to the nature of soil itself, but also must understand the effect of reasonable environmental impacts upon the excavated material. In the author’s experience, some notable environmental impacts include the following:

- excess water content (sources may include runoff, seepage, aquifers, precipitation)
- location of water table
- pore pressure
- ability of water to drain from the soil (hydraulic conductivity)
- deficiency in soil moisture content (degree of saturation)
- protective systems employed and their effectiveness
- vibration sources
- how the soil was deposited (wind, gravity, water)
- forces of gravity (i.e., weight of soil)
- time of exposure
- trench depth
- loss of static coefficient of friction and/or normal force
- presence of previously disturbed soil
- weather cycles including freeze–thaw and frost action
- tension cracks (size, depth, and location of)
- fissures

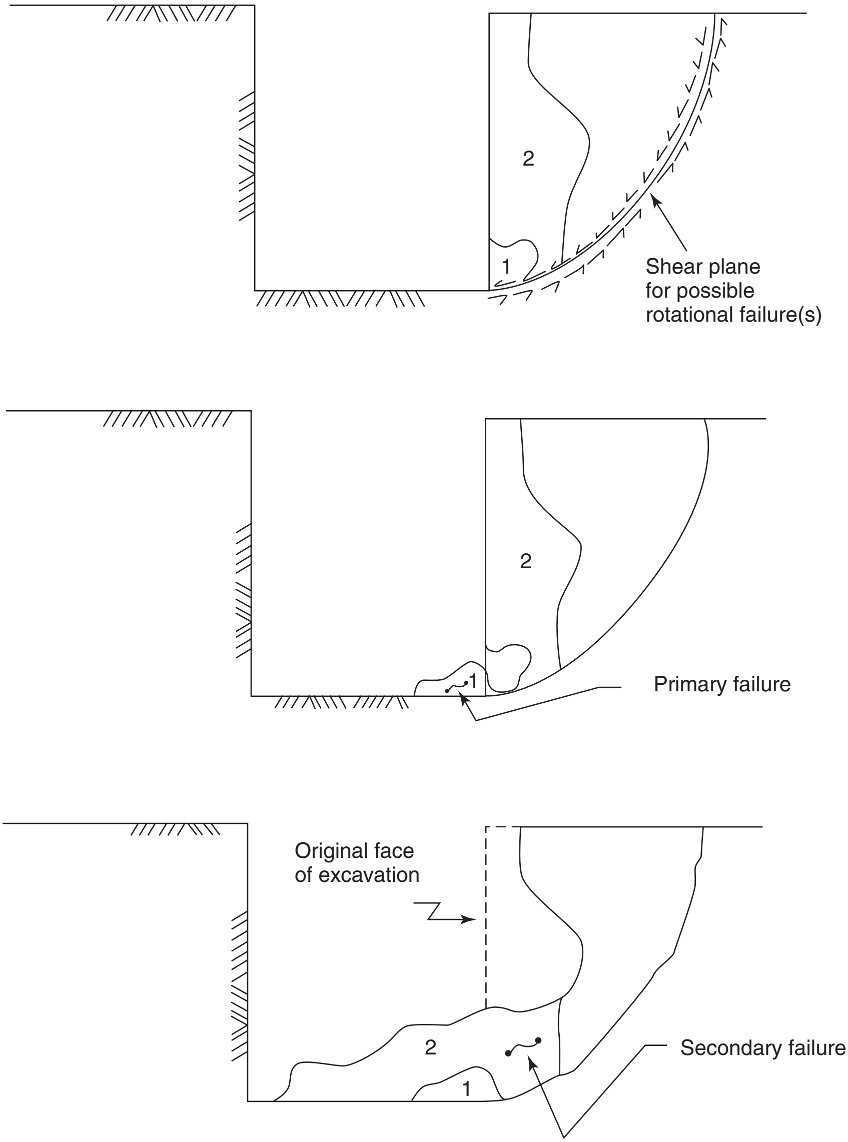


Figure 8.2 Excavated materials may fail repeatedly (often called a secondary collapse).

- stress history of the soil (i.e., has it been over burdened?)
- slickensides
- piping effects, seepage flow nets, hydraulic gradients
- boiling, heaving, or quick conditions
- fiber content (if organic material or peat)
- secondary compression rate and total potential

- reaction to water (i.e., swelling, settlement, compressibility, decomposition)
- confining pressure from the sides (i.e., as in a three-dimensional model – effective stress)
- imposed surcharge loads (point and/or continuous loading)
- decomposition or chemical breakdown of soil depending on binder for cementation of particles
- inclination of stratum, layering, bedding planes, and stratification of multiple soil deposit(s)
- consolidation, compaction, deformation, and density of the material.

These are just a few of the issues that a competent person must be aware of based simply on the material to be excavated.

Put simply, managing for occupational safety of excavation work begins with looking at the initial stability of the excavated soil itself, then it quickly focuses on those factors that may further influence stability. However, this still does not complete the analysis or the job of a competent person.

In addition, work incidental to excavation work brings its own risks and hazards. One example is the possibility of encountering utility lines (overhead or underground), and another is exposure of the general public to work zone activity. This topic of incidental hazards in excavation work is developed further below.

However, with proper planning, training, inspection, practical knowledge, and skill, all one needs to be safe in excavation work is common sense and a thumb. Some say that if it is predictable, it is preventable.

This chapter is comprised of the following subsections:

- Hazard Identification and Federal OSHA Regulation
- Soil Types
- Basic Soil Mechanics Theory
- Testing and Soil Classification Systems
- Protective Systems.

8.2
Hazard Identification and Federal OSHA Regulation

The majority of the Federal OSHA's regulation on excavation safety is found in Subpart ''P'' of 29 CFR 1926 (Sections 650, 651, and 652 – with Appendices A–F) (see *OSHA.gov* for more information). As excerpted below, in section 29 CFR 1926.651 are standards covering (a) surface encumbrances, (b) underground installations, (c) access and egress, (d) vehicular traffic, (e) falling loads, (f) mobile equipment warning systems, (g) hazardous atmospheres, (h) water accumulation, (i) stability of adjacent structures (j) loose rock and soil, (k) inspections, and (l) walkways.

1926.651(a) Surface encumbrances. All surface encumbrances that are located so as to create a hazard to employees shall be removed or supported, as necessary, to safeguard employees.

1926.651(b) Underground installations.

1926.651(b) (1) The estimated location of utility installations, such as sewers and telephone, fuel, electric, and water lines, or any other underground installations that reasonably may be expected to be encountered during excavation work, shall be determined prior to opening an excavation.

1926.651(b) (2) Utility companies or owners shall be contacted within established or customary local response times, advised of the proposed work, and asked to establish the location of the utility underground installations prior to the start of actual excavation. When utility companies or owners cannot respond to a request to locate underground utility installations within 24 h (unless a longer period is required by state or local law), or cannot establish the exact location of these installations, the employer may proceed, provided that the employer does so with caution, and provided that detection equipment or other acceptable means to locate utility installations is used.

1926.651(b) (3) When excavation operations approach the estimated location of underground installations, the exact location of the installations shall be determined by safe and acceptable means.

1926.651(b) (4) While the excavation is open, underground installations shall be protected, supported, or removed as necessary to safeguard employees.

1926.651(c) Access and egress.

1926.651(c) (1) Structural ramps.

1926.651(c) (1) (i) Structural ramps that are used solely by employees as a means of access or egress from excavations shall be designed by a competent person. Structural ramps used for access or egress of equipment shall be designed by a competent person qualified in structural design, and shall be constructed in accordance with the design.

1926.651(c) (1) (ii) Ramps and runways constructed of two or more structural members shall have the structural members connected together to prevent displacement.

1926.651(c) (1) (iii) Structural members used for ramps and runways shall be of uniform thickness.

1926.651(c) (1) (iv) Cleats or other appropriate means used to connect runway structural members shall be attached to the bottom of the runway or shall be attached in a manner to prevent tripping.

1926.651(c) (1) (v) Structural ramps used in lieu of steps shall be provided with cleats or other surface treatments o the top surface to prevent slipping.

1926.651(c) (2) Means of egress from trench excavations. A stairway, ladder, ramp, or other safe means of egress shall be located in trench excavations that are 4 ft (1.22 m) or more in depth so as to require no more than 25 ft (7.62 m) of lateral travel for employees.

1926.651(d) Exposure to vehicular traffic. Employees exposed to public vehicular traffic shall be provided with, and shall wear; warning vests or other suitable garments marked with or made of reflectorized or high-visibility material.

1926.651(e) Exposure to falling loads. No employee shall be permitted underneath loads handled by lifting or digging equipment. Employees shall be required to stand away from any vehicle being loaded or unloaded to avoid being struck by any spillage or falling materials. Operators may remain in the cabs of vehicles being loaded or unloaded when the vehicles are equipped, in accordance with 1926.601(b) (6), to provide adequate protection for the operator during loading and unloading operations.

1926.651(f) Warning system for mobile equipment. When mobile equipment is operated adjacent to an excavation, or when such equipment is required to approach the edge of an excavation, and the operator does not have a clear and direct view of the edge of the excavation, a warning system shall be utilized such as barricades, hand or mechanical signals, or stop logs. If possible, the grade should be away from the excavation.

1926.651(g) Hazardous atmospheres:

1926.651(g) (1) Testing and controls. In addition to the requirements set forth in subparts D and E of this part (29 CFR 1926.50—1926.107) to prevent exposure to harmful levels of atmospheric contaminants and to assure acceptable atmospheric conditions, the following requirements shall apply:

1926.651(g) (1) (i) Where oxygen deficiency (atmospheres containing less than 19.5% oxygen) or a hazardous atmosphere exists or could reasonably be expected to exist, such as in excavations in landfill areas or excavations in areas where hazardous substances are stored nearby, the atmospheres in the excavation shall be tested before employees enter excavations greater than 4 ft (1.22 m) in depth.

1926.651(g) (1) (ii) Adequate precautions shall be taken to prevent employee exposure to atmospheres containing less than 19.5% oxygen and other hazardous atmospheres. These precautions include providing proper respiratory protection or ventilation in accordance with subparts d and e of this part, respectively.

1926.651(g) (1) (iii) Adequate precautions shall be taken, such as providing ventilation, to prevent employee exposure to an atmosphere containing a concentration of a flammable gas in excess of 20% of the lower flammable limit of the gas.

1926.651(g) (1) (iv) When controls are used that are intended to reduce the level of atmospheric contaminants to acceptable levels, testing shall be conducted as often as necessary to ensure that the atmosphere remains safe.

1926.651(g) (2) Emergency rescue equipment.

1926.651(g) (2) (i) Emergency rescue equipment, such as breathing apparatus, a safety harness and line, or a basket stretcher, shall be readily available where hazardous atmospheric conditions exist or may reasonably be expected to develop during work in an excavation. This equipment shall be attended when in use.

1926.651(g) (2) (ii) Employees entering bell-bottom pier holes, or other similar deep and confined footing excavations, shall wear a harness with a lifeline securely attached to it. The lifeline shall be separate from any line used to handle materials, and shall be individually attended at all times while the employee wearing the lifeline is in the excavation.

1926.651(h) Protection from hazards associated with water accumulation.

1926.651(h) (1) Employees shall not work in excavations in which there is accumulated water, or in excavations in which water is accumulating, unless adequate precautions have been taken to protect employees against the hazards posed by water accumulation. The precautions necessary to protect employees adequately vary with each situation, but could include special support or shield systems to protect from cave-ins, water removal to control the level of accumulating water, or use of a safety harness and lifeline.

1926.651(h) (2) If water is controlled or prevented from accumulating by the use of water removal equipment, the water removal equipment, and operations shall be monitored by a competent person to ensure proper operation.

1926.651(h) (3) If excavation work interrupts the natural drainage of surface water (such as streams), diversion ditches, dikes, or other suitable means shall be used to prevent surface water from entering the excavation and to provide adequate drainage of the area adjacent to the excavation. Excavations subject to runoff from heavy rains will require an inspection by a competent person and compliance with paragraphs (h) (1) and (h) (2) of this section.

1926.651(i) Stability of adjacent structures.

1926.651(i) (1) Where the stability of adjoining buildings, walls, or other structures is endangered by excavation operations, support systems such as shoring, bracing, or underpinning shall be provided to ensure the stability of such structures for the protection of employees.

1926.651(i) (2) Excavation below the level of the base or footing of any foundation or retaining wall that could be reasonably expected to pose a hazard to employees shall not be permitted except when:

1926.651(i) (2) (i) A support system, such as underpinning, is provided to ensure the safety of employees and the stability of the structure; or

1926.651(i) (2) (ii) The excavation is in stable rock; or

1926.651(i) (2) (iii) A registered professional engineer has approved the determination that the structure is sufficiently removed from the excavation so as to be unaffected by the excavation activity; or

1926.651(i) (2) (iv) A registered professional engineer has approved the determination that such excavation work will not pose a hazard to employees.

1926.651(i) (3) Sidewalks, pavements, and appurtenant structures shall not be undermined unless a support system or another method of protection is provided to protect employees from the possible collapse of such structures.

1926.651(j) Protection of employees from loose rock or soil.

1926.651(j) (1) adequate protection shall be provided to protect employees from loose rock or soil that could pose a hazard by falling or rolling from an excavation face. Such protection shall consist of scaling to remove loose material; installation of protective barricades at intervals as necessary on the face to stop and contain falling material; or other means that provide equivalent protection.

1926.651(j) (2) Employees shall be protected from excavated or other materials or equipment that could pose a hazard by falling or rolling into excavations. Protection shall be provided by placing and keeping such materials or equipment at least 2 ft (0.61 m) from the edge of excavations, or by the use of retaining devices that are sufficient to prevent materials or equipment from falling or rolling into excavations, or by a combination of both if necessary.

1926.651(k) Inspections.

1926.651(k) (1) Daily inspections of excavations, the adjacent areas, and protective systems shall be made by a competent person for evidence of a situation that could result in possible cave-ins, indications of failure of protective systems, hazardous atmospheres, or other hazardous conditions. An inspection shall be conducted by the competent person prior to the start of work and as needed throughout the shift. Inspections shall also be made after every rainstorm or other hazard-increasing occurrence. These inspections are only required when employee exposure can be reasonably anticipated.

1926.651(k) (2) Where the competent person finds evidence of a situation that could result in a possible cave-in, indications of failure of protective systems, hazardous atmospheres, or other hazardous conditions, exposed employees shall be removed from the hazardous area until the necessary precautions have been taken to ensure their safety.

1926.651(l) Walkways shall be provided where employees or equipment are required or permitted to cross over excavations. Guardrails which comply with 1926.502(b) shall be provided where walkways are 6 ft (1.8 m) or more above lower levels.

Now that the regulations have been presented, we shall discuss some of the engineering and physics of excavation and trenching and their impact on loss prevention.

8.3 Soil Types

First, it is important to understand that OSHA defines and classifies types of soil differently than a geotechnical engineer might. Generally, OSHA offers four classifications (Stable Rock and Types A, B, and C) that define soil based on the stability of the material. Based on the author's experience and training, a geotechnical engineer, on the other hand, defines or classifies soil based on six distinct ranges of particle size (from smallest to largest: Clay, Silt, Sand, Gravel, Cobble, and Boulder) and also by several special properties that a soil may exhibit.

The OSHA soil classification system is a method of categorizing soil and rock deposits in a hierarchy of Stable Rock, Type A, Type B, and Type C, in decreasing order of stability. OSHA looks to the soil's stability as measured in term of ''unconfined compressive strength'' (In units of ton ft^{-2} or kg m^{-2}).

More specifically, OSHA soil classifications are determined based on an analysis (by a competent person) of the properties and performance characteristics of the deposits and the characteristics of the deposits and the environmental conditions of exposure. This is the load per unit area at which a soil will fail in compression.

It can be determined by laboratory testing, or estimated in the field using a pocket penetrometer, by thumb penetration tests, and other methods. OSHA uses a conservative standard.

OSHA offers these four types of soil classification as listed below:

- *Stable rock* means natural solid mineral matter that can be excavated with vertical sides and remains intact while exposed.
- *Type A* means cohesive soils with an unconfined, compressive strength of 1.5 t ft^{-2} (tsf) (144 kPa) or greater. Examples of cohesive soils are clay, silty clay, sandy clay, clay loam and, in some cases, silty clay loam and sandy clay loam. Cemented soils such as caliche and hardpan are also considered Type A.
- However, no soil is Type A if:
  - the soil is fissured; or
  - the soil is subject to vibration from heavy traffic, pile driving, or similar effects; or
  - the soil has been previously disturbed; or
  - the soil is part of a sloped, layered system where the layers dip into the excavation on a slope of four horizontal to one vertical (4H:1V) or greater; or
  - the material is subject to other factors that would require it to be classified as a less stable material.
- *Type B* means:
  - cohesive soil with an unconfined compressive strength greater than 0.5 tsf (48 kPa) but less than 1.5 tsf (144 kPa); or
  - granular cohesionless soils including angular gravel (similar to crushed rock), silt, silt loam, sandy loam, and, in some cases, silty clay loam and sandy clay loam.
  - previously disturbed soils except those which would otherwise be classed as Type C soil.
  - soil that meets the unconfined compressive strength or cementation requirements for Type A, but is fissured or subject to vibration; or
  - dry rock that is not stable; or
  - material that is part of a sloped, layered system where the layers dip into the excavation on a slope less steep than four horizontal to one vertical (4H:1V), but only if the material would otherwise be classified as Type B.
- *Type C* means:
  - cohesive soil with an unconfined compressive strength of 0.5 tsf (48 kPa) or less; or
  - granular soils including gravel, sand, and loamy sand; or
  - submerged soil or soil from which water is freely seeping; or
  - submerged rock that is not stable, or
  - material in a sloped, layered system where the layers dip into the excavation or a slope of four horizontal to one vertical (4H:1V) or steeper.

Geotechnical engineers use the following ranges of particle or grain sizes (from largest to smallest) to classify soil:

Boulders 12 *in* + (30.48 cm+)	
Cobbles 3 in–12 in (6.45–30.48 cm)	
Gravel	
	Coarse $^3/_4 - 3$ in (1.9–6.45 cm)
Fine No. 4 sieve[a] to $^3/_4$ in	
Sand	
	Coarse No. 10 sieve to No. 4 sieve
	Medium No. 40 sieve to No. 10 sieve
	Fine No. 200 sieve to No. 40 sieve
Silt[b] and clay "fines" passing No. 200 sieve[c]	

[a] Sieve No. reflects the number of openings per square inch.
[b] Fine-grained soils cannot be visually divided between silt and clay, but are distinguishable by plasticity characteristics determined in Atterberg limit tests and other field tests.
[c] Clay exhibits cohesion (a stickiness due to particle–particle attraction) and is significantly smaller than silt particles that exhibit "apparent cohesion" or a sandcastle effect in the author's experience when particles are bound together by moisture.

Finally, the properties of several "special" soil and rock deposits as listed below are outlined:

1) loess
2) expansive soils
3) organic soils and peat
4) colluvium and talus
5) shales and degradable materials
6) caliche and cemented sands
7) sensitive clays and sands.

8.3.1 Loess

Loess is a subset of a geological classification called Aeolian. It is a collapsible material that was deposited during glacial periods and is commonly found in many parts of the world and the central United States plus portions of Washington, Oregon, Idaho, and Alaska. It is a uniform, wind-blown (i.e., Aeolian) soil consisting primarily of silt-sized particles. The apparent cohesion of the material is the result of calcareous clay binder that holds the silt particles together. The clay coating and wind-blown formation create a very loose soil structure with little true particle-to-particle contact, especially at low confining pressures. In contrast, another type of wind-deposited (non-glacial) Aeolian soil is called "Dune."

Collapsible soils such as loess are generally classified as soils that undergo a relatively significant and sudden decrease in volume when water is introduced because, upon wetting, the cohesion in the soil is lost and large settlements can occur even if the loading remains constant. Saturation of the loess material "softens" the calcareous clay binder and greatly decreases strength. This, in turn, may lead to slope instability and accelerated erosion.

Collapsible soils usually exist in the ground at relatively low values of dry unit weight and moisture content. Under these conditions the materials are moderately strong and exhibit a slight but characteristic apparent cohesion. In their natural state, such soils can support moderate loads and undergo relatively small settlements.

For loess (and also for man-made fills), relatively low standard penetration tests (SPTs) as measured in *N* values can be used as an indicator of the potential for collapse. The engineering behavior of loess is affected by whether the loess is sandy, silty, or clayey. Silty loess is characterized as being extremely erodible; sandy and clayey loess types are collapsible but are much less erodible than silty loess. Hydrometer testing should be performed on the material passing a No. 200 sieve to evaluate the quantity of sand, silt, and clay.

8.3.2 Expansive Soils

Expansive soils are typically clayey soils that undergo large volume changes in direct response to moisture changes in the soil. Unlike collapsible soils, expansive soils tend to increase in volume (i.e., swell) as the moisture content of the soil is increased and decrease in volume (i.e., shrink) as the moisture content of the soil is decreased. Although the expansion potential of a soil can be related to many factors (e.g., soil structure and fabric, environmental conditions), it is primarily controlled by the clay mineralogy.

Soils that contain low-plasticity kaolinite will tend to exhibit a lower shrink/swell potential than soils containing high-plasticity montmorillonite. Expansive soils are found throughout the United States; however, damage caused by expansive clays is most prevalent in California, Wyoming, Colorado, and Texas, where the climate is considered to be semi-arid and periods of intense rainfall are followed by long periods of drought. This pattern of wet and dry cycles results in periods of extensive near-surface drying and desiccation crack formation. During intense precipitation, water enters the deep cracks, permitting the soil to swell; upon drying, the soil will shrink. This weather pattern results in cycles of swelling and shrinking that can be detrimental to the performance of pavements, slabs on-grade, and retaining walls and excavation.

Deep-seated volume changes in expansive soils are rare. More common are volume changes within the upper few meters of a soil deposit. These upper few meters are more likely to be affected by seasonal moisture content changes due to climatic changes.

In the field, the presence of surface desiccation cracks and/or fissures in a clay deposit is an indication of expansion potential. Experience has indicated that the most problematic expansive near-surface soils are typically highly plastic, stiff, fissured overconsolidated clays.

8.3.3 Organic Soils and Peat

Organic soils (i.e., organic clays and organic silts) present similar engineering challenges to soft silts and clays, including low undrained shear strengths and high compressibility. In addition, organic silts and clays undergo significant secondary (or creep) deformations. Such long-term, continuous deformation can present significant maintenance issues for embankments and other structures that may be founded over such materials. Like other organic soils, peats also undergo significant secondary deformations.

Organic soils and peats are evidenced during subsurface exploration based on the presence of decaying vegetative matter and a strong odor. Typically, the materials are greenish, dark gray, or black in color and can have very fibrous structures with wood fragments and plant remains.

Living coral and coralline debris are generally found in tropical regions where the water temperature exceeds 20 °C. Coral is a term commonly used for the group of animals which secrete an outer skeleton composed of calcium carbonate, and which generally grow in colonies. The term ‘‘coral reef’’ is often applied to large concentrations of such colonies which form extensive submerged tracts around tropical coasts and islands. In general, coralline soils deposited after the breakdown of the reef, typically by wave action, are thin (a few meters thick) and form a veneer upon cemented materials (limestones, sandstones, etc.).

Concerning geological classification, because the granular coralline and algal materials are derived from organisms which vary in size from microscopic shells to large coralheads several meters in diameter, the fragments are broadly graded and range in size from boulders to fine-grained muds. Similarly, the shape of these materials varies from sharp, irregular fragments to well-rounded particles. Coralline deposits are generally referred to as ‘‘biogenic materials’’ by geologists. When cemented, they may be termed ‘‘reef rock,’’ or ‘‘beachrock,’’ or other names which imply an origin through cementation of particles into a hard, coherent material.

8.3.4 Colluvium and Talus

Colluvium and talus are weathered materials that migrate and accumulate on the sides and at the toe of slopes. Fine-grained material with rock fragments is called colluviumand coarse-grained material with boulders is called talus. Colluvium (colluvial soil) and talus are generally relatively loose deposits found near the base of slopes and may accumulate in valleys, swales, or other low-lying topographic features.

Colluvial soils generally result from a two-stage process of (i) in-place weathering of the parent rock and (ii) subsequent migration downslope primarily by gravity through creep. Colluvium commonly consists of rock fragments in a heterogeneous clayey to sandy matrix. Shale and other degradable materials include claystone and mudstone that degrade to the parent soil material upon contact with water and air; slope stability will progressively decrease over time.

Talus, like colluvium, also accumulates at the base of slopes, but generally consists of the mechanically weathered granular component of the parent rock. Talus slopes are commonly characterized as well-graded boulders to sand- or silt-sized particles

In general, colluvium occurs in temperate and humid environments whereas talus predominantly occurs in arid and semi-arid regions. The characteristics of the parent bedrock and the climate in which the weathering and migration/transportation take place determine the characteristics of the colluvium and talus deposits.

Accumulations of colluvium and talus on the sideslopes and at the base of sideslopes are often associated with slope stability problems. Cut slopes often have to be made in colluvium and talus deposits located near the base of a slope. In many cases, the cut slope often exposes the colluvium/talus material. Because these materials typically form by migration and sliding along the slope, they are often only marginally stable in their natural state. Therefore, the cut slopes made in these deposits tend to disrupt the natural equilibrium, thus requiring aggressive monitoring and maintenance.

8.3.5
Shales and Degradable Materials

Many rock types are prone to degradation when exposed to the cyclic wet–dry and freeze–thaw weathering processes. Rock types that are particularly susceptible to degradation due to these processes are poorly indurated shale exhibiting a high clay content.

Shale initially exhibits rock-like characteristics; it has the potential to degrade to soil-sized particles. The gradual but ultimate degradation of the rock to the original parent soil material can occur within minutes or after several years of exposure to air and/or water. Shale, the most common member of this family of materials, can generically be considered to include claystone, siltstone, and mudstone.

The degradation can take the form of swelling, weakening, and ultimately disintegration. The effect of degradation on slope stability can range from surficial sloughing and gradual retreat of the face, to catastrophic slope failures resulting from the significant loss of strength. In sedimentary rock formations comprising alternating beds of resistant sandstone and relatively degradable shale, the weathering process can develop overhangs in the sandstone and produce a rockfall hazard.

8.3.6
Caliche and Cemented Sands

Cemented sands are naturally occurring granular materials that have a cementing material either in the void space between individual grains or at the points of grain-to-grain contact. The cementing agent may be soluble or insoluble.

The primary consequence of cementation is to increase strength and reduce compressibility relative to uncemented materials. Weakly cemented sands can form due to grain-to-grain point-contact welding as a result of aging or from a clay–silt binder that accompanies a wind-blown dune deposit. At the other extreme, the cemented sand can be characterized as weak sandstone, where a carbonate bond may occur at the grain contacts. Regardless of the method of origin, cementation results in stiff, but brittle, load-deformation behavior.

The result of the cementing action is that the sand exhibits a true cohesion (i.e., a component of shear strength that is independent of confining pressure). As a result, cemented sands are usually ''stronger'' than uncemented sands. At testing, the high blow counts make material appear as dense sand; penetration tests meet refusal; rock coring equipment may be necessary, but note that simply adding water may cause dissolution of binder soil to make it brittle while it initially appears to be very strong.

8.3.7 Sensitive Clays and Sands

Quick clays, material transported and deposited by oceans, are characterized by their great sensitivity or strength reduction upon disturbance. All quick clays are of marine origin. Because of their brittle nature, collapse occurs at relatively small strains. Slopes in quick clays can fail without large movements. Sensitive clays are confined to specific geographic regions. The metastable structure of sensitive clays is established when relatively low-plasticity clays are deposited in brackish (i.e., salty) waters in a flocculated particle orientation.

The resulting high void ratio soil structure, due to the edge-to-face alignment of the clay plates, is stable although the soils in the deposit have a high natural moisture content and a moderately high liquidity index. In this state, the clay is weak, but not likely sensitive. The sensitive characteristics are introduced when fresh water is leached through the uplifted deposit, replacing the brackish water with the fresh water.

Extremely loose sands that tend to strain soften can be subject to catastrophic static liquefaction. Fortunately, the natural occurrence of these materials is very rare, as they are usually associated with hydraulically filled structures. In the author's experience, sensitive clays, although naturally occurring, are generally confined to specific geographic regions of the world where there was extensive glaciation followed by isostatic uplift.

8.4 Basic Soil Mechanics Theory

Soil is made up of solids (mineral particles), water, air, and sometimes organics. Basically, soil failures are due to a loss of shear strength in the soil. A shear force in a soil tends to cause adjacent particles to slide relative to each other. The shear strength of a soil is a measure of its resistance to internal sliding.

Three primary factors contribute to a soil's shear strength:

1) cohesion (true and/or apparent)
2) internal friction (at peak or residual strength)
3) consolidation with or without a possible a resulting increase in pore pressure that impacts the effective stress, (i.e., the pressure of contact between soil particles.)

True cohesion is a particle-to-particle attraction between clay particles. Apparent cohesion is present when moisture binds the particles of silt or sand together (sometimes called a >sandcastle effect). Soils composed of strictly sands, gravel, cobbles, and boulders are said to be cohesionless and (with rare exceptions) do not exhibit true cohesion.

To understand and identify the difference between true cohesion (clay) and apparent cohesion (silt and sand) is cardinal to understanding basic soil mechanics theory. However, cohesion is not the only engineering property that provides a soil's stability through resistance to shear.

Friction also provides shear strength to soil. Soil friction is based in part on variables such as size, shape, texture, particle interlock, roughness, density, rigidity, and moisture content of the solid materials.

The principal modes of failure in soil or rock are (i) rotation on a curved slip surface approximated by a circular arc, (ii) translation on a planar surface whose length is large compared with depth below ground, and (iii) displacement of a wedge-shaped mass along one or more planes of weakness. Other modes of failure include toppling of rock slopes, falls, block slides, lateral spreading, earth and mud flow in clayey and silty soils, and debris flows in coarse-grained soils.

In homogeneous cohesive soils, the critical failure surface is usually deep and rotational whereas shallow surface sloughing and sliding are more typical in homogeneous cohesionless soils. In granular soils, instability usually does not extend significantly below the excavation provided that seepage forces are controlled. Rather, instability may be seen in terms of sloughing, bulging, or weak pockets.

Often the weakest soil on the job is that which has most recently been dug (i.e., the spoils). Likewise, the more recently the soil has been disturbed, the less likely it is to be close to its virgin state. OSHA requires spoils to be kept a minimum of 2 ft from the edge of the excavation.

The mineral particles do not fit together perfectly, and the voids between them are filled with water or air. The water or air in the voids is called pore-water or pore-air. A soil whose pores are filled entirely with water is called a saturated soil. Most soils below the water table are saturated.

A soil whose pores are filled with both water and air is called an unsaturated soil. The compacted fill material in embankments is an unsaturated soil. The combined pressure of the weight of a soil and any applied load is called total stress. The pressure carried by the soil particles in contact with each other is called effective stress. The pressure of the water in the voids is called pore-water pressure. When the pore-water pressure increases, effective stress is reduced. When the pore-water pressure decreases, effective stress is increased.

This principle can be expressed as

$$\text{effective stress} = \text{total stress} - \text{pore-water pressure}$$

Note that the terms ‘‘stress’’ and ‘‘pressure’’ refer to force per unit area and can generally be used interchangeably. Thus, ‘‘total stress’’ is the same as ‘‘total pressure’’ and ‘‘effective stress’’ is the same as ‘‘effective pressure.’’

When a load is applied to a dry soil, the soil particles are forced closer together. In contrast, when a load is applied to a saturated soil, the soil particles cannot move closer together, since water fills the voids between them. Instead, the load is transferred to the pore-water. This creates excess pore-water pressure and causes the water to flow into soil where the pore-water pressure is lower. As the water flows out of the voids, the soil particles are able to consolidate, to move closer together. Consolidation occurs rapidly at first, because there is a high pressure gradient between the loaded soil and the non-loaded soil. As the pressure-gradient decreases, consolidation slows.

An in-flow of water affects the equilibrium of the slope in two ways. It increases shear forces by adding mass and it decreases shear strength by raising pore-water pressure. Measurements of pore-water pressure can be used to predict slope failure and can assist the design of retention and drainage systems.

Plane failures tend to occur in layered soils or rock where one layer is weaker than the others.

The rate at which excess pore-water pressure dissipates and the soil consolidates depends on the permeability of the soil. In a granular soil such as sand, the pore-water flows away quickly and consolidation occurs quickly. In less permeable soils such as clays, dissipation of the excess pore-water pressure is much slower, and consolidation can take many months or even years. As consolidation occurs, the volume occupied by the soil decreases and settlement occurs. As pore-water pressure increases, effective stress and shear strength are reduced.

Ground water increases the chances of failure: it adds mass to the block and it decreases shear strength. When soils are unsaturated, a basic analysis can be carried out using total stress. When soil is saturated, a more accurate analysis is obtained using effective stress.

Consolidation refers to the time-dependent decrease in the volume of a soil due to the dissipation of pore pressures within the soil mass. Shortly after a soil is first loaded (i.e., after immediate or undrained distortional settlements have occurred), the stresses are transmitted to the pore fluid in the soil mass, resulting in excess pore pressures. As these pore pressures dissipate with time, the load is gradually transferred to effective stress within the soil skeleton. The resulting increase in effective stress results in a decrease in volume that causes settlements.

- **Gravels, sands, and non-plastic silts**: These soils consolidate rapidly under load and do not typically present settlement problems.
- **Plastic silt-clay mixtures**: Soft silts and clays are more compressible than stiff silts and clays. Settlement may continue long after construction is complete.

- **Organic soils**: These soils are very compressible and biodegradable, and can result in large settlements that occur for many years.

Failure planes in rock occur along zones of weakness or discontinuities (fissures, joints, faults) and bedding planes (strata). The orientation and strength of the discontinuities are the most important factors influencing the stability of rock slopes.

Discontinuities can develop or strength can change as a result of the following environmental factors:

1) chemical weathering
2) freezing and thawing of water/ice in joints
3) joint patterns
4) tectonic movements
5) slope of excavation
6) increase of water pressures within discontinuities
7) alternate wetting and drying (especially expansive shales)
8) increase in tensile stresses due to differential erosion.

With natural slopes, failures will occur when the rupturing force exceeds the resisting force. Natural slopes and man-made slopes may have an imbalance of forces that could be caused by one or more of the following factors:

1) A change in slope profile that adds driving weight at the top or decreases resisting force at the base. Examples include steepening of the slope and undercutting of the toe.
2) An increase in groundwater pressure, resulting in a decrease in frictional resistance in cohesionless soil or swell in cohesive material. Groundwater pressures may increase through the saturation of a slope from rainfall or snowmelt, seepage from an artificial source, or rise of the water table,
3) Progressive decrease in shear strength of the soil or rock mass caused by weathering, leaching, mineralogical changes, opening and softening of fissures, or continuing gradual shear strain (creep).
4) Vibrations induced by equipment, road traffic earthquakes, blasting, or pile-driving.
5) If water enters the fissures, the strength of the clay will decrease progressively. Therefore, the long-term stability of slopes excavated in cohesive soils is normally more critical than the short-term stability. When excavations are open over a long period and water is accessible, there is potential for swelling and loss of strength with time.

Finally, lateral forces due to frost action are difficult to predict and may achieve high values. Materials such as silts and clayey silts, described as CL, MH, ML, and OL in the Unified Soil Classification System (USCS), where C = inorganic clay, M = inorganic silt, O = organic silts and silt-clays, L = low plasticity, and H = high plasticity), are frost susceptible, and will exert excessive pressure if proper precautions are not taken to curb frost; swelling pressures may also be exerted by clays of high plasticity (CH).

8.5
Testing and Soil Classification Systems

This section discusses common tests used to classify soils. It begins with those tests covered in OSHA's Appendix A of 29 CFR 1926.652 (Subpart ''P'' Excavation) (OSHA, 2912b). The material reproduced immediately below speaks for itself:

1) **Classification of soil and rock deposits**: Each soil and rock deposit shall be classified by a competent person as Stable Rock, Type A, Type B, or Type C in accordance with the definitions set forth in paragraph (b) of this Appendix.
2) **Basis of classification**: The classification of the deposits shall be made based on the results of at least one visual and at least one manual analysis. Such analyses shall be conducted by a competent person using tests described in paragraph (d) below, or in other recognized methods of soil classification and testing such as those adopted by ASTM International [formerly the American Society for Testing and Materials (ASTM)], or the US Department of Agriculture textural classification system.
3) **Visual and manual analyses**: The visual and manual analyses, such as those noted as being acceptable in paragraph (d) of this Appendix, shall be designed and conducted to provide sufficient quantitative and qualitative information as may be necessary to identify properly the properties, factors, and conditions affecting the classification of the deposits.
4) **Layered systems**: In a layered system, the system shall be classified in accordance with its weakest layer. However, each layer may be classified individually where a more stable layer lies under a less stable layer.
5) **Reclassification**: If, after classifying a deposit, the properties, factors, or conditions affecting its classification change in any way, the changes shall be evaluated by a competent person. The deposit shall be reclassified as necessary to reflect the changed circumstances.
6) **Acceptable visual and manual tests**:
   a. **Visual tests**: Visual analysis is conducted to determine qualitative information regarding the excavation site in general, the soil adjacent to the excavation, the soil forming the sides of the open excavation, and the soil taken as samples from excavated material.
      i. Observe samples of soil that are excavated and soil in the sides of the excavation. Estimate the range of particle sizes and the relative amounts of the particle sizes. Soil that is primarily composed of fine-grained material is cohesive material. Soil composed primarily of coarse-grained sand or gravel is granular material.
      ii. Observe soil as it is excavated. Soil that remains in clumps when excavated is cohesive. Soil that breaks up easily and does not stay in clumps is granular.
      iii. Observe the side of the opened excavation and the surface area adjacent to the excavation. Crack-like openings such as tension cracks could indicate fissured material. If chunks of soil spall off a vertical side, the

soil could be fissured. Small spalls are evidence of moving ground and are indications of potentially hazardous situations.

iv. Observe the area adjacent to the excavation and the excavation itself for evidence of existing utility and other underground structures, and to identify previously disturbed soil.

v. Observe the opened side of the excavation to identify layered systems. Examine layered systems to identify if the layers slope towards the excavation. Estimate the degree of slope of the layers.

vi. Observe the area adjacent to the excavation and the sides of the opened excavation for evidence of surface water, water seeping from the sides of the excavation, or the location of the level of the water table.

vii. Observe the area adjacent to the excavation and the area within the excavation for sources of vibration that may affect the stability of the excavation face.

b. **Manual tests**: Manual analysis of soil samples is conducted to determine quantitative as well as qualitative properties of soil and to provide more information in order to classify soil properly.

i. **Plasticity**: Mold a moist or wet sample of soil into a ball and attempt to roll it into threads as thin as $^1/_8$ in (3.2 mm) in diameter. Cohesive material can be successfully rolled into threads without crumbling. For example, if at least a 2 in (50 mm) length of $^1/_8$ in thread can be held on one end without tearing, the soil is cohesive.

ii. **Dry strength**: If the soil is dry and crumbles on its own or with moderate pressure into individual grains or fine powder, it is granular (any combination of gravel, sand, or silt). If the soil is dry and falls into clumps which break up into smaller clumps, but the smaller clumps can only be broken up with difficulty, it may be clay in any combination with gravel, sand, or silt. If the dry soil breaks into clumps which do not break up into small clumps and which can only be broken with difficulty, and there is no visual indication that the soil is fissured, the soil may be considered unfissured.

iii. **Thumb penetration**: The thumb penetration test can be used to estimate the unconfined compressive strength of cohesive soils. [This test is based on the thumb penetration test described in ASTM Standard D2488 – ''Standard Recommended Practice for Description of Soils (Visual–Manual Procedure).''] Type A soils with an unconfined compressive strength of 1.5 tsf can be readily indented by the thumb; however, they can be penetrated by the thumb only with very great effort. Type C soils with an unconfined compressive strength of 0.5 tsf can be easily penetrated several inches by the thumb, and can be molded by light finger pressure. This test should be conducted on an undisturbed soil sample, such as a large clump of spoil, as soon as practicable after excavation to keep to a minimum the effects of exposure to drying influences. If the excavation is later exposed to wetting influences (rain, flooding), the classification of the soil must be changed accordingly.

iv. **Other strength tests**: Estimates of unconfined compressive strength of soils can also be obtained by use of a pocket penetrometer or a hand-operated shearvane (Figures 8.3–8.5).
A pocket penetrometer is used for obtaining the shear strength of cohesive, non-gravelly soils on field exploration or construction sites. Commercial penetrometers are available which read unconfined compressive strength directly. The tool is used as an aid to obtaining uniform classification of soils. It does not replace other field tests or laboratory tests.
A shearvane device is used for obtaining rapid approximations of shear strength of cohesive, non-gravelly soils on field exploration. These can be used on the ends of Shelby tubes, penetration samples, and block samples from test pits or sides of test pits. The device is used in uniform soils and does not replace laboratory tests.
v. **Drying test**: The basic purpose of the drying test is to differentiate between cohesive material with fissures, unfissured cohesive material, and granular material. The procedure for the drying test involves drying a sample of soil that is ~1 in (2.54 cm) thick and 6 in (15.24 cm) in diameter until it is thoroughly dry:
   I) If the sample develops cracks as it dries, significant fissures are indicated.
   II) Samples that dry without cracking are to be broken by hand. If considerable force is necessary to break a sample, the soil has significant cohesive material content. The soil can be classified as

Figure 8.3 Pocket penetrometer (Courtesy OSHA.)

Figure 8.4 Shearvane. (Courtesy OSHA.)

Figure 8.5 Shearvane application. (Courtesy OSHA.)

an unfissured cohesive material and the unconfined compressive strength should be determined.

III) If a sample breaks easily by hand, it is either a fissured cohesive material or a granular material. To distinguish between the two, pulverize the dried clumps of the sample by hand or by stepping on them. If the clumps do not pulverize easily, the material is cohesive with fissures. If they pulverize easily into very small fragments, the material is granular.

Soil shear strength is influenced by many factors, including the cohesion, friction, effective stress state, mineralogy, packing arrangement of the soil particles, soil hydraulic conductivity, rate of loading, stress history, sensitivity, and other variables.

Other recognized tests for use in classifying soil include the following:

- **Vane shear apparatus**: *In situ* vane shear measurements are especially useful in very soft soil deposits where much of the strength may be lost by disturbance during sampling. It should not be used in stiff clays or in soft soils containing gravel, shells, wood, and so on. The main equipment components are the torque assembly, which includes a gear reduction device capable of producing a constant angular rotation of $1–6^{\circ}\,\text{min}^{-1}$, a calibrated proving ring with a dial gauge for torque measurement within 5%, a means of measuring angular rotation in degrees, and thrust bearings to support the vane at the ground surface. Procedures for the vane shear test and methods of interpretation are described in ASTM Standard D2573, Field Vane Shear Test in Cohesive Soil.
- **Atterberg limits**: The Atterberg limits of a fine-grained (i.e., clayey or silty) soil represent the moisture content at which the behavior of the soil changes. The tests for the Atterberg limits (Figure 8.6) are referred to as index tests because they serve as an indication of several physical properties of the soil, including strength, permeability, compressibility, and shrink/swell potential. These limits also provide a relative indication of the plasticity of the soil, where plasticity refers to the ability of a silt or clay to retain water without changing state from a semi-solid to a viscous liquid. In geotechnical engineering practice, the Atterberg limits generally refers to the liquid limit (LL), plastic limit (PL), and shrinkage limit (SL). These limits are defined as follows:

  Liquid limit (LL): This upper limit represents the moisture content at which any increase in moisture content will cause a plastic soil to behave as a liquid. The LL is defined as the moisture content at which a standard groove cut in a remolded sample will close over a distance of ½ in (1.27 cm) at 25 blows of the LL device.

  Plastic limit (PL): This limit represents the moisture content at which the transition between the plastic and semisolid state of a soil. The PL is defined as the moisture content at which a thread of soil just crumbles when it is carefully rolled out to a diameter of 3.2 mm.

  Shrinkage limit (SL): The moisture content corresponding to the behavior change between the semisolid to solid state of the soil. The SL is also defined as the moisture content at which any further reduction in moisture content will not result in a decrease in the volume of the soil.

A measure of a soils plasticity is the plasticity index (PI), which as calculated as $\text{PI} = \text{LL} - \text{PL}$.

The PI is a useful index since numerous engineering correlations have been developed relating PI to clay soil properties, including undrained and drained

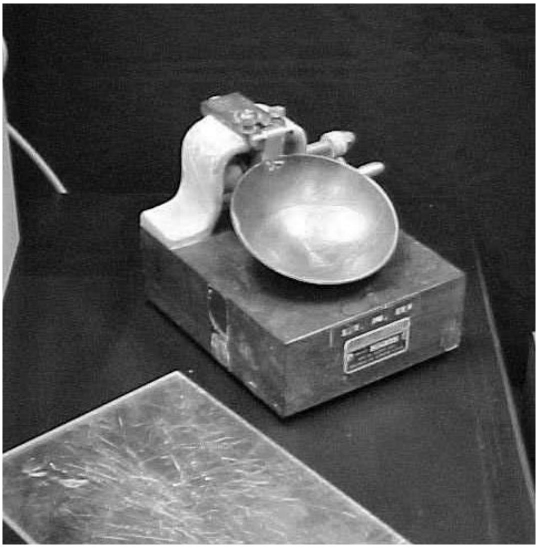


Figure 8.6 Atterberg limit apparatus. (Courtesy OSHA.)

strength and compression index. Results are typically presented on Casagrande's plasticity chart.

8.5.1 Standard Penetration Test (SPT)

The most commonly used *in situ* test is the standard penetration test (SPT) (American Association of State Highway Transportation Officials (AASHTO) T206, ASTM D1586). It measures resistance to the penetration of a standard sampler in borings.

The SPT is a simple and rugged test suitable for most soil types except gravel and is usually performed using a conventional geotechnical drill rig. SPTs are recommended for essentially all subsurface investigations since a disturbed sample can be obtained for baseline soil property interpretation. The test provides a rough index of the relative strength and compressibility of the soil in the vicinity of the test. The method is rapid and, when tests are properly conducted in the field, they yield useful data, although there are many factors which can affect the results. A guide to consistency of fine-grained soils is presented in Table 8.1.

The test is covered under ASTM Standard D1586, which requires the use of a standard 2 in (5.08 cm) outer diameter split barrel sampler, driven by a 140 lb (63.5 kg) hammer dropping 30 in (76 cm) in free fall over a 12 in (30.5 cm) distance after disregarding the first 6 in (15.24 cm).

The SPT should not be relied upon in soils containing coarse gravel, cobbles, or boulders, because the sampler can become obstructed, resulting in high and

Table 8.1 Guide to consistency of fine-grained soils[a].

N value (blows per foot)	SPT strength	Estimated unconfined compressive strength (tsf)
<2	Very soft	<0.25 (extruded between fingers when squeezed)
2–4	Soft	0.25–0.50 (molded by light finger pressure)
4–8	Medium	0.50–1.00 (molded by strong finger pressure)
8–15	Stiff	1.00–2.00 (readily indented by thumb but penetrated with great effort)
15–30	Very stiff	2.00–4.00 (readily indented by thumbnail)
>30	Hard	>4.00 (indented with difficulty by thumbnail)

[a]OSHA designates Type A soil in the range >1.5 tsf, Type B soil in the range 1.5—0.5 tsf, and Type C soil in the range <0.5 tsf.
[a]Lindeburg (1990).

unconservative *N* values. The test should not be relied on for cohesionless silts because dynamic effects at the sampler tip can lead to erroneous strength and compressibility evaluations. The test also has little meaning in soft and sensitive clays.

Since the SPT is highly dependent upon the equipment and operator performing the test, it is often difficult to obtain repeatable results. The main factors affecting the SPT results are summarized below.

8.5.1.1 Procedures Which May Affect the Measured *N* Values

1) SPT is only partially made in original soil.
2) Sludge may be trapped in the sampler and compressed in the borehole as the sampler is driven, increasing the blow count.
3) Not seating the sampler spoon on undisturbed material.
4) Driving of *N* values is increased in sands and reduced in cohesive soils.
5) Spoon above the bottom of the casing.
6) Failure to maintain sufficient hydrostatic head in boring.
7) The water table in the borehole must be at least equal to the piezometric level in the sand, otherwise the sand at the bottom of the borehole may be transformed into a loose state.
8) Attitude of operators – blow counts for the same soil using the same rig can vary, depending on who is operating the rig,
9) Higher blow counts usually result from an overdriven sampler.
10) Higher blow counts result when gravel plugs the sampler.
11) Resistance of loose sand could be highly overestimated.
12) High *N* values may be recorded for loose sand when sampling below the groundwater table. Hydrostatic pressure causes sand to rise and plug the casing.

13) Low blow count may result for dense sand since sand is loosened by overwashing.
14) Drilling method – drilling technique (e.g., cased holes versus mud-stabilized holes) may result in different *N* values for the same soil.
15) Energy delivered per blow is not uniform.
16) Free fall of the drive weight
17) Using more than 1 1/2 turns of rope around the drum and/or using wire cable will restrict the fall of the drive weight.
18) Not using the correct weight: the driller frequently supplies drive hammers with weight, with the weights varying from the standard by as much as 10 lb (4.54 kg).
19) Weight does not strike the drive cap concentrically: impact energy is reduced, increasing *N* values.
20) Not using a guide rod.
21) Not using a good tip on the sampling spoon: if the tip is damaged and reduces the opening or increases the end area, the *N* value can be increased.
22) Use of drill rods heavier than standard: with heavier rods more energy is absorbed by the rods, causing an increase in the blow count.
23) Not recording blow counts and penetration accurately.
24) Incorrect drilling techniques: drilling procedures which seriously disturb the soil will affect the *N* value, for example, drilling with cable tool equipment.
25) Inadequate supervision.
26) Frequently a sampler will be impeded by gravel or cobbles, causing a sudden increase in blow count.
27) Using too large a pump: too high a pump capacity will loosen the soil at the base of the hole, causing a decrease in blow count.

8.5.2 Cone Penetrometer

A more controlled test is the cone penetrometer test, in which a cone-shaped tip is jacked from the surface of the ground to provide a continuous resistance record.

8.5.3 Other Types of Shear Tests (Laboratory)

For cohesive soils and in the case of fine-grained soils which have low permeability, total stress strength parameters are used. As will be discussed, the values of cohesion and friction may be determined from laboratory unconfined compression tests or undrained triaxial tests. As previously discussed, shear strength correlations with SPTs, shearvanes, and cone penetration tests may also be used.

Many types and variations of shear tests have been developed. In most of these tests, the rate of deformation is controlled and the resulting loads are measured. In some tests, total stress parameters are determined, whereas in others effective

Figure 8.7 Direct shear testing box. (Courtesy Geo-Testing Express.)

stress strength parameters are obtained. The following are the most widely used laboratory testing procedures:

1) **Direct shear tests**: The apparatus and procedures for direct shear testing are discussed in ASTM D 3080. A thin soil sample is placed in a shear box consisting of two parallel blocks (Figure 8.7). The lower block is fixed while the upper block is moved parallel to it in a horizontal direction. The soil fails by shearing along a plane assumed to be horizontal. This test is relatively easy to perform. Consolidated-drained (CD) tests can be performed on soils of low permeability in a short period of time as compared with the triaxial test. However, the stress, strain, and drainage conditions during shear are not as accurately understood or controlled as in the triaxial test.
   This test can be used to evaluate the drained strength of natural materials by shearing the sample at a slow enough rate to ensure reasonably that no pore-water pressures develop.
   For designs involving geosynthetics, the strengths of the interface between the soil and geosynthetic or between the geosynthetic and geosynthetic are often necessary parameters. Direct shear machines have been modified to test the shear strength of various interfaces, as described in ASTM D5321.
2) **Triaxial compression test**: Triaxial systems today use electronic instrumentation to provide continuous monitoring of test data. A cylindrical sample is confined by a membrane and lateral pressure is applied; pore-water drainage is controlled through tubing connected to porous discs at the ends of the sample Force is measured using a force transducer or load cell that is typically mounted outside the triaxial cell, but more advanced systems have incorporated the transducer within the testing cell to reduce rod-friction effects.
   Results of this test are reported graphically and known as the *Mohr's circle diagram*. A triaxial test yields the following Coulomb equation: shear stress (plotted on the *y*-axis) equals cohesion plus friction [where friction is a function of the normal stress (or downward applied force) and the tested material's internal angle of friction).

Triaxial shear strength testing is performed on undisturbed samples of cohesive soils. Triaxial testing can be generally classified as: (i) unconsolidated-undrained (UU); (ii) consolidated-undrained (CU); and (iii) consolidated-drained (CD).

a. **UU or quick test (Q)**: In the UU test, the initial water content of the test specimen is not permitted to change during shearing of the specimen. The shear strength of soil as determined in UU tests corresponds to total stress, and is applicable only to situations where little consolidation or drainage can occur during shearing.
b. **CU or R test**: In the CU test, complete consolidation of the test specimen is permitted under the confining pressure, but no drainage is permitted during shear. A minimum of three tests is required to define strength parameters of cohesion and internal angle of friction, although four test specimens are preferable with one serving as a check. Specimens must, as a general rule, be completely saturated before application of the deviator stress.
c. **CD or S test**: In the CD test, complete consolidation of the test specimen is permitted under the confining pressure and drainage is permitted during shear. The rate of strain is controlled to prevent the build-up of pore pressure in the specimen. A minimum of three tests is required. CD tests are generally performed on well-draining soils. For slow-draining soils, several weeks may be required to perform a CD test.

During shearing, load is applied at a rate slow enough to allow drainage of pore-water and no build-up of pore-water pressure. Again, the time required to conduct this test in low-permeability soil may be as long as several months; therefore, it is not common to conduct this test on low-permeability soils. This test models the long-term (drained) condition in soil.

Factors affecting triaxial test results include appropriately correcting for membrane stiffness, piston friction, and filter drains, whenever applicable. The shear strength of soft, sensitive soils is greatly affected by sample disturbance. The laboratory-measured shear strength of disturbed samples will be lower than the in-place strength in the case of UU tests. In the case of CU or CD tests, the strength may be higher because of the consolidation permitted.

8.6 Protective Systems

Typically there are three types of excavation protective systems referenced in OSHA's excavation standard, namely sloping (includes benching), shoring, and shielding (often referred to as the 3-S′). Based on a competent person's classification of the respective soil together with relevant tabulated data, a determination can be made regarding the proper selection of a protective system.

With respect to an employer's duty to provide excavation protective systems, OSHA offers the following:

1926.652(a) Protection of employees in excavations:

Table 8.2 Maximum allowable slopes (OSHA, 2012b).

Soil or rock type	Maximum allowable slope (H:V)[a] for excavations less than 20 ft deep[a]
Stable rock	Vertical (90°)[b]
Type A[c]	$^3/_4$:1 (53°)
Type B	1:1 (45°)
Type C	1 $^1/_2$:1 (34°)

[a] Sloping or benching for excavations greater than 20 ft deep shall be designed by a registered professional engineer.
[b] Numbers in parentheses next to maximum allowable slopes are angles expressed in degrees from the horizontal. Angles have been rounded off.
[c] A short-term maximum allowable slope of $^1/_2$H:1V (63°) is allowed in excavations in Type A soil that are 12 ft (3.67 m) or less in depth. Short-term maximum allowable slopes for excavations greater than 12 ft (3.67 m) in depth shall be $^3/_4$H:1V (53°).

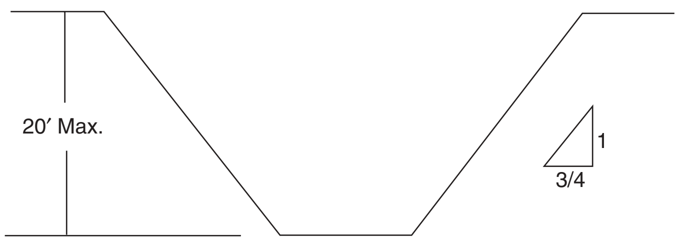
– 1926.652(a) (1) Each employee in an excavation shall be protected from cave-ins by an adequate protective system designed in accordance with paragraph (b) or (c) of this section, except when:
– 1926.652(a) (1) (i) Excavations are made entirely in stable rock; or
– 1926.652(a) (1) (ii) Excavations are less than 5 ft (1.52 m) in depth and examination of the ground by a competent person provides no indication of a potential cave-in.

8.6.1 Slope Configurations (OSHA 29 CFR 1926)

All slopes stated below are expressed as the horizontal to vertical ratio. Maximum allowable slopes are given in Table 8.2 (OSHA, 2012b).

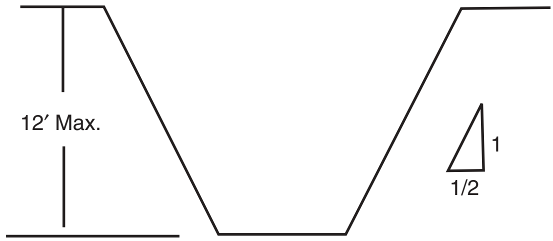
8.6.1.1 Excavations Made in Type A Soil

1) All simple slope excavations 20 ft or less in depth shall have a maximum allowable slope of $^3/_4$:1.



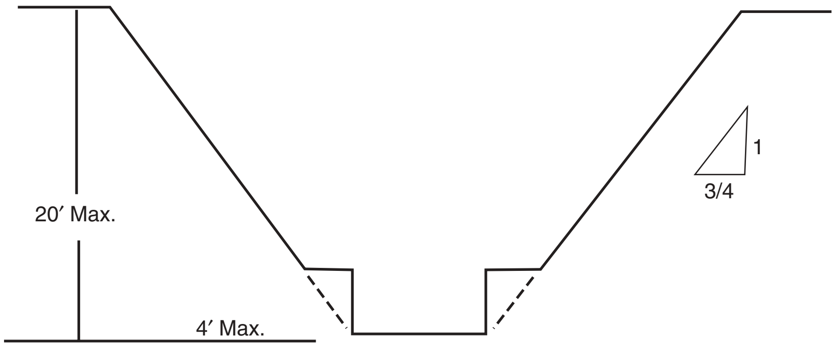
Simple slope – general

Exception: Simple slope excavations which are open 24 h or less (short term) and which are 12 ft or less in depth shall have a maximum allowable slope of $^1/_2$:1.

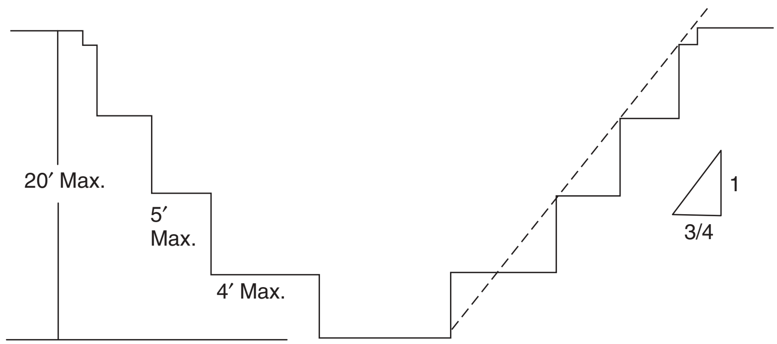


Simple slope – short term

2) All benched excavations 20 ft or less in depth shall have a maximum allowable slope of $^3/_4$:1 and maximum bench dimensions as follows:

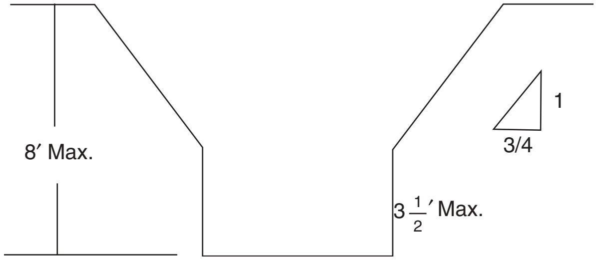


Simple bench

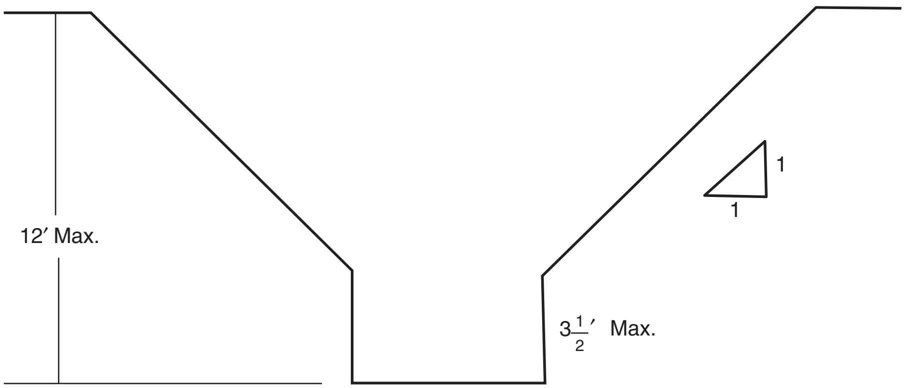


Multiple benches

3) All excavations 8 ft or less in depth which have unsupported vertically sided lower portions shall have a maximum vertical side of 3 1/2 ft.

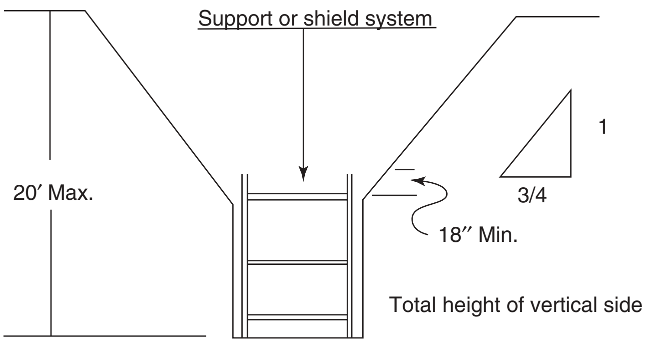


Unsupported vertically sided lower portion – maximum 8 ft in depth
All excavations more than 8 ft but not more than 12 ft in depth with unsupported vertically sided lower portions shall have a maximum allowable slope of 1:1 and a maximum vertical side of 3 1/2 ft.



Unsupported vertically sided lower portion – maximum 12 ft in depth

4) All excavations 20 ft or less in depth which have vertically sided lower portions that are supported or shielded shall have a maximum allowable slope of 3/4:1. The support or shield system must extend at least 18 in above the top of the vertical side.

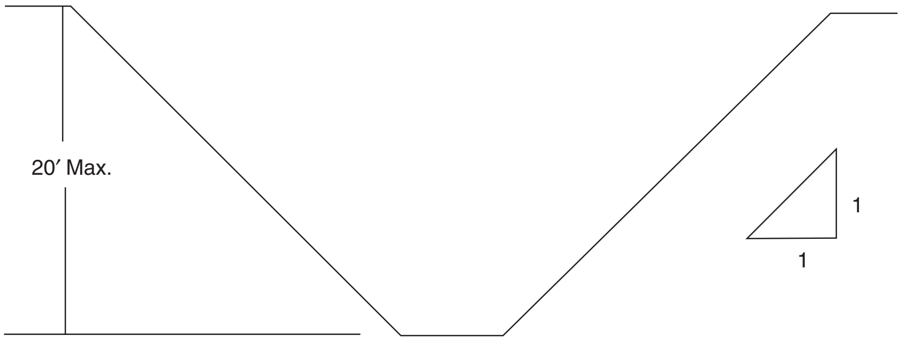


Supported or shielded vertically sided lower portion

5) All other simple slope, compound slope, and vertically sided lower portion excavations shall be in accordance with the other options permitted under §1926.652(b).

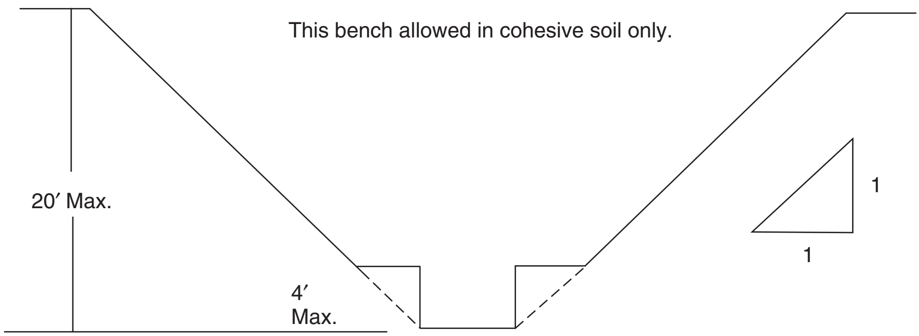
8.6.1.2 Excavations Made in Type B Soil (OSHA 29 CFR 1926)

1) All simple slope excavations 20 ft or less in depth shall have a maximum allowable slope of 1:1.

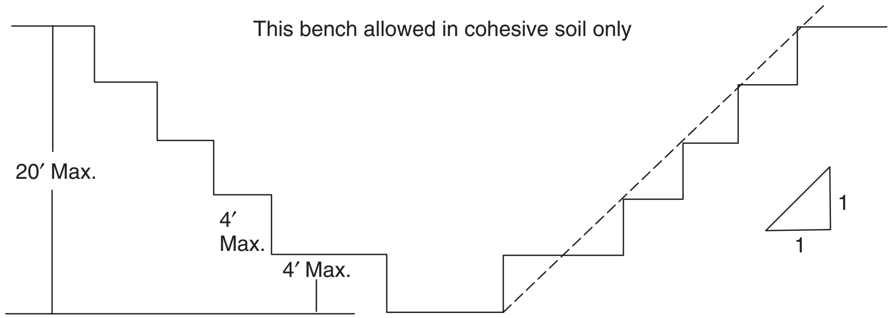


Simple slope

2) All benched excavations 20 ft or less in depth shall have a maximum allowable slope of 1:1 and maximum bench dimensions as follows:



Single bench

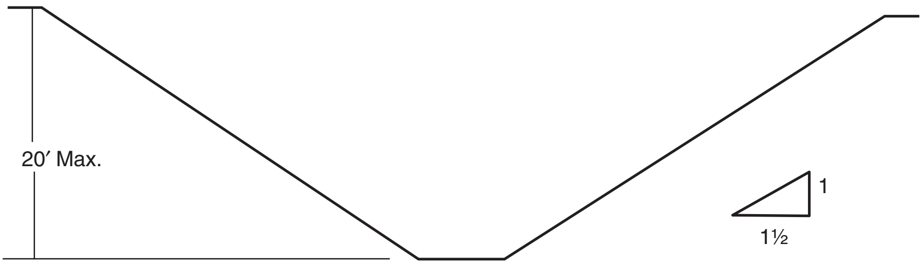


Multiple bench

3) All excavations 20 ft or less in depth which have vertically sided lower portions shall be shielded or supported to a height at least 18 in above the top of the vertical side. All such excavations shall have a maximum allowable slope of 1:1.
4) All other sloped excavations shall be in accordance with the other options permitted in §1926.652(b).

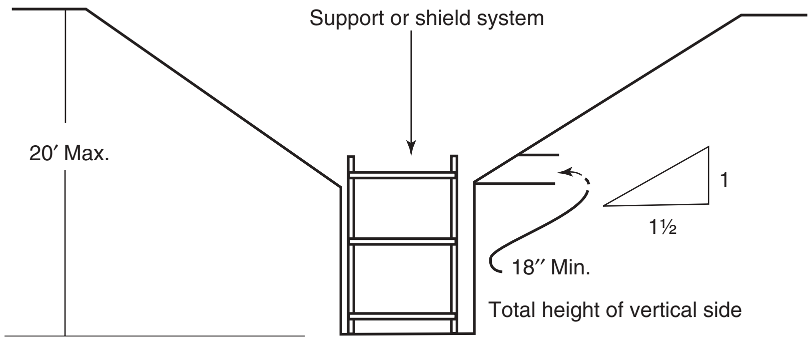
8.6.1.3 Excavations Made in Type C Soil (OSHA 29 CFR 1926)

1) All simple slope excavations 20 ft or less in depth shall have a maximum allowable slope of $1\,^1/_2$:1.



Simple slope

2) All excavations 20 ft or less in depth which have vertically sided lower portions shall be shielded or supported to a height at least 18 in. above the top of the vertical side. All such excavations shall have a maximum allowable slope of $1\,^1/_2$:1.



Vertically sided lower portion

3) All other sloped excavations shall be in accordance with the other options permitted in §1926.652(b).

8.6.2
Timber Shoring – Appendix C to OSHA 29 CFR 1926.652

1) **Scope**: This Appendix contains information that can be used when timber shoring is provided as a method of protection from cave-ins in trenches that do not exceed 20 ft (6.1 m) in depth (see Figure 8.8). This Appendix must be used when design of timber shoring protective systems is to be performed in accordance with 1926.652(c) (1). Other timber shoring configurations; other systems of support such as hydraulic and pneumatic systems; and other protective systems such as sloping, benching, shielding, and freezing systems must be designed in accordance with the requirements set forth in 1926.652(b) and 1926.652(c).
2) **Soil classification**: In order to use the data presented in this Appendix, the soil type or types in which the excavation is made must first be determined using the soil classification method set forth in Appendix A of subpart P of this part.

The OSHA standard calls out two types of shoring materials, namely Oak and Douglas Fir.

8.6.3
Aluminum Hydraulic Shoring – Appendix D to OSHA 29 CFR 1926.652

1) **Scope**: This Appendix contains information that can be used when aluminum hydraulic shoring is provided as a method of protection against cave-ins in trenches that do not exceed 20 ft (6.1 m) in depth (see Figure 8.9). This Appendix must be used when design of the aluminum hydraulic protective system cannot be performed in accordance with 1926.652(c) (2).

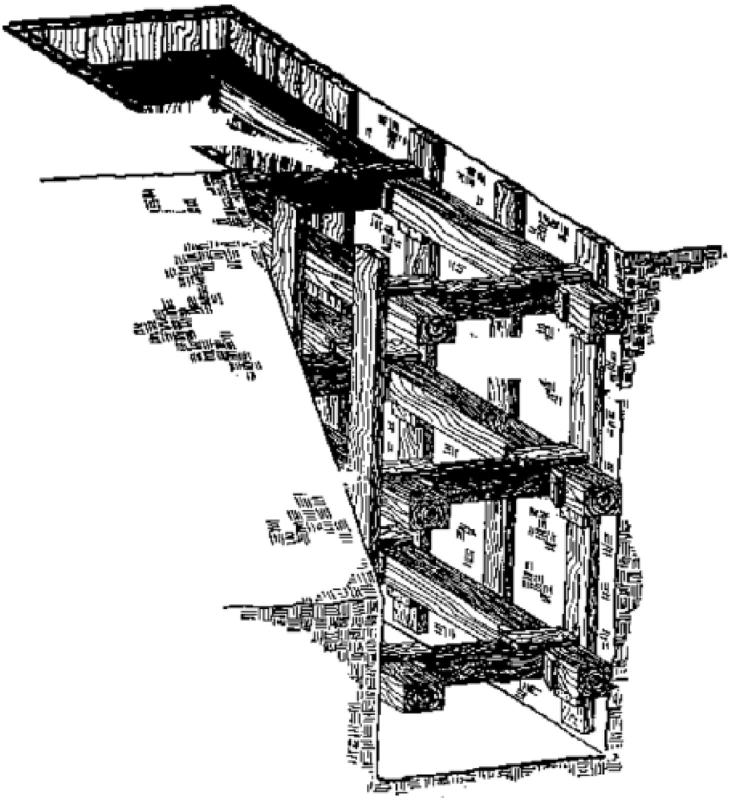


Figure 8.8 Timber shoring. (Courtesy OSHA.)

2) **Soil classification**: In order to use data presented in this Appendix, the soil type or types in which the excavation is made must first be determined using the soil classification method set forth in Appendix A of subpart P of part 1926.

8.6.4 Alternatives to Timber Shoring – Appendix E of OSHA 29 CFR 1926.652

For use of alternative excavation protection to timber shoring, such as pneumatic systems, screw jacks, or trench boxes (see Figure 8.10), consult manufacturers' tabulated data and relevant operation and maintenance manuals.

Glossary (Taken in Part from *Standard Handbook for Civil Engineers* (Ricketts, Loftin, and Merritt, 2003) and OSHA 29 CFR 1926 (OSHA, 2012b))

Actual slope the slope to which an excavation face is excavated.

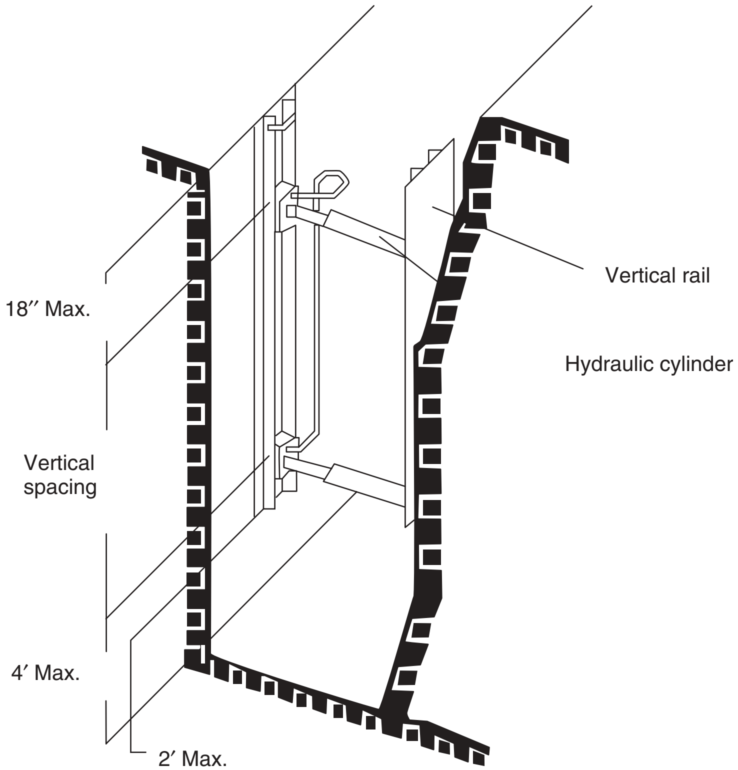


Figure 8.9 Aluminum hydraulic shoring (OSHA, 2012b).

Aeolian material transported and deposited by wind such as loess and other collapsible soils. Collapse of the structure of these soils can cause a reduction of cohesion and a rise in pore pressure.

Aluminum hydraulic shoring a pre-engineered shoring system comprised of aluminum hydraulic cylinders (cross braces) used in conjunction with vertical rails (uprights) or horizontal rails (wales). Such a system is designed specifically to support the sidewalls of an excavation and prevent cave-ins.

Anisotropic soil A soil mass having different properties in different directions at any given point, referring primarily to stress–strain or permeability characteristics.

Apparent cohesion where water binds particle soil particle together, that is, sandcastle effects.

Bell-bottom pier hole a type of shaft or footing excavation, the bottom of which is made larger than the cross-section above to form a bell shape.

Benching (benching system) a method of protecting employees from cave-ins by excavating the sides of an excavation to form one or a series of horizontal levels or steps, usually with vertical or near-vertical surfaces between levels.

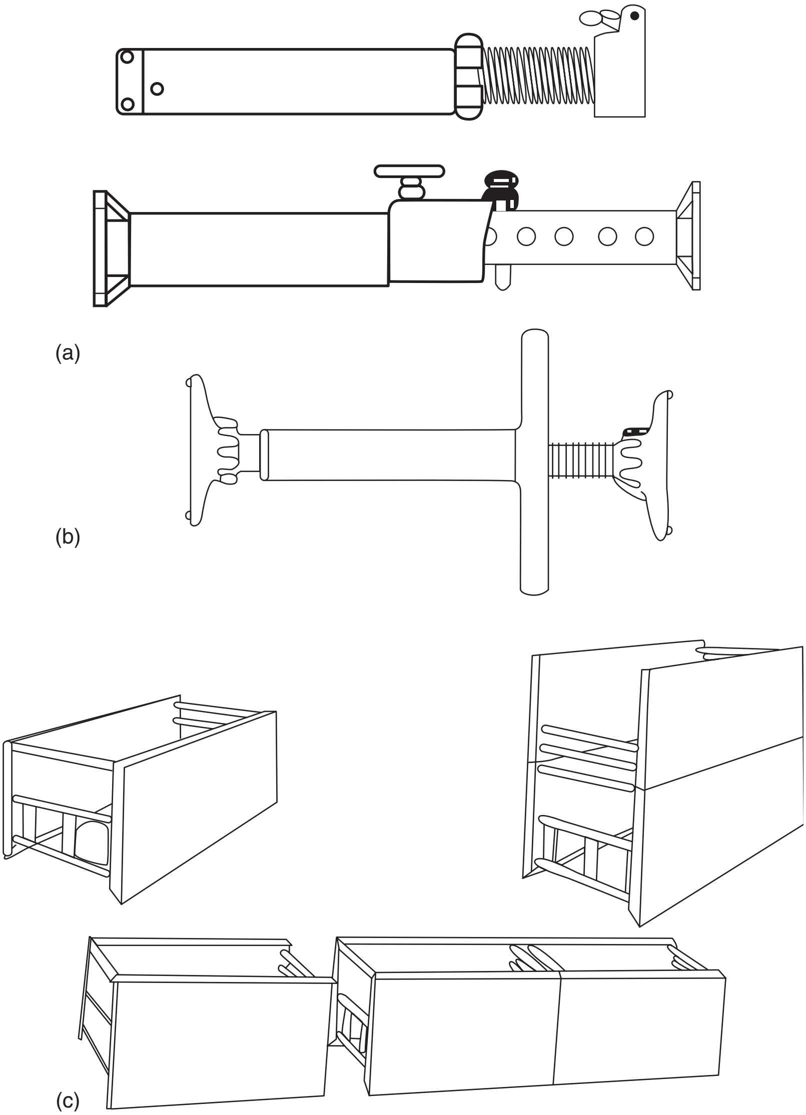


Figure 8.10 Alternative excavation protection to timber shoring. (a) Pneumatic/hydraulic shoring; (b) trench jacks (screw jacks); (c) trench shields.

Capillary stresses Pore-water pressures less than atmospheric values produced by surface tension of pore-water acting on the meniscus formed in void spaces between soil particles.

Cave-in the separation of a mass of soil or rock material from the side of an excavation, or the loss of soil from under a trench shield or support system, and its sudden movement into the excavation, either by falling or sliding, in sufficient quantity so that it could entrap, bury, or otherwise injure and immobilize a person.

Cemented soil a soil in which the particles are held together by a chemical agent, such as calcium carbonate, such that a hand-sized sample cannot be crushed into powder or individual soil particles by finger pressure.

Cohesion a particle-to-particle attraction from the electrochemical attraction between the particles.

Cohesive soil clay (fine-grained soil), or soil with a high clay content, which has cohesive strength. Cohesive soil does not crumble, can be excavated with vertical side slopes, and is plastic when moist. This material is hard to break up when dry, and exhibits significant cohesion when submerged. Cohesive soils include clayey silt, sandy clay, silty clay, clay, and organic clay.

Colluvial material transported and deposited by gravity.

Competent person one who is capable of identifying existing and predictable hazards in the surroundings, or working conditions which are unsanitary, hazardous, or dangerous to employees, and who has authorization to take prompt corrective measures to eliminate them.

Consolidation refers to the time-dependent decrease in volume of a soil due to the dissipation of pore pressures within the soil mass.

Cross braces the horizontal members of a shoring system installed perpendicular to the sides of the excavation, the ends of which bear against either uprights or wales.

Desiccation The process of shrinkage or consolidation of the fine-grained soil produced by an increase in effective stresses in the grain skeleton accompanying the development of capillary stresses in the pore-water.

Distress soil that is in a condition where a cave-in is imminent or is likely to occur. Distress is evidenced by such phenomena as the development of fissures in the face of or adjacent to an open excavation; the subsidence of the edge of an excavation; the slumping of material from the face or the bulging or heaving of material from the bottom of an excavation; the spalling of material from the face of an excavation; and raveling, that is, small amounts of material such as pebbles or small clumps of material suddenly separating from the face of an excavation and trickling or rolling down into the excavation.

Dry soil soil that does not exhibit visible signs of moisture content.

Effective stress The net stress across points of contact of soil particles, generally considered as equivalent to the total stress minus the pore-water pressure.

Excavation any man-made cut, cavity, trench, or depression in an earth surface, formed by earth removal.

Excess pore pressures that increment of pore-water pressures greater than hydrostatic values, produced by consolidation stresses in compressible materials or by shear strain.

Faces or sides the vertical or inclined earth surfaces formed as a result of excavation work.

Failure the breakage, displacement, or permanent deformation of a structural member or connection so as to reduce its structural integrity and its supportive capabilities.

Fissured a soil material that has a tendency to break along definite planes of fracture with little resistance, or a material that exhibits open cracks, such as tension cracks, in an exposed surface.

Glacial till material transported and deposited by glaciers.

Granular soil gravel, sand, or silt (coarse-grained soil) with little or no clay content. Granular soil has no cohesive strength. Some moist granular soils exhibit apparent cohesion. Granular soil cannot be molded when moist and crumbles easily when dry.

Hazardous atmosphere an atmosphere which by reason of being explosive, flammable, poisonous, corrosive, oxidizing, irritating, oxygen deficient, toxic, or otherwise harmful, may cause death, illness, or injury.

Hydrostatic pore pressures Pore-water pressures or groundwater pressures exerted under conditions of no flow where the magnitude of the pore pressures increases linearly with depth below the ground surface.

Isotropic soil a soil mass having essentially the same properties in all directions at any given point, referring primarily to stress–strain or permeability characteristics.

Kickout the accidental release or failure of a cross brace.

Over-consolidation the condition that exists if a soil deposit has been subjected to an effective stress greater than the existing overburden pressure.

Layered system two or more distinctly different soil or rock types arranged in layers. Micaceous seams or weakened planes in rock or shale are considered layered.

Loam a combination soil made up from clay, silt, and/or sand.

Maximum allowable slope the steepest incline of an excavation face that is acceptable for the most favorable site conditions as protection against cave-ins, and expressed as the ratio of horizontal distance to vertical rise (H:V).

Moist soil a condition in which a soil looks and feels damp. Moist cohesive soil can easily be shaped into a ball and rolled into small-diameter threads before crumbling. Moist granular soil that contains some cohesive material will exhibit signs of cohesion between particles.

Piezometer a device installed for measuring the pressure head of pore-water at a specific point within the soil mass.

Piping the movement of soil particles as the result of unbalanced seepage forces produced by percolating water, leading to the development of boils or erosion channels.

Plastic a property of a soil which allows the soil to be deformed or molded without cracking or appreciable volume change.

Protective system a method of protecting employees from cave-ins, from material that could fall or roll from an excavation face or into an excavation, or from the collapse of adjacent structures. Protective systems include support systems, sloping and benching systems, shield systems, and other systems that provide the necessary protection.

Pyroclastic material transported and deposited by volcanoes.

Ramp an inclined walking or working surface that is used to gain access to one point from another, and is constructed from earth or from structural materials such as steel or wood.

Registered professional engineer a person who is registered as a professional engineer in the State where the work is to be performed. However, a professional engineer registered in any State is deemed to be a ''registered professional engineer'' within the meaning of this standard (OSHA's 29 CFR 1926) when approving designs for ''manufactured protective systems'' or ''tabulated data'' to be used in interstate commerce.

Saturated soil a soil in which the voids are filled with water. Saturation does not require flow. Saturation, or near saturation, is necessary for the proper use of instruments such as a pocket penetrometer or sheer vane.

Sheeting the members of a shoring system that retain the earth in position and in turn are supported by other members of the shoring system.

Shield (shield system) a structure that is able to withstand the forces imposed on it by a cave-in and thereby protect employees within the structure. Shields can be permanent structures or can be designed to be portable and moved along as work progresses. Additionally, shields can be either premanufactured or job-built in accordance with OSHA 1926.652(c) (3) or (c) (4). Shields used in trenches are usually referred to as ''trench boxes'' or ''trench shields.''

Shoring (shoring system) a structure such as a metal hydraulic, mechanical, or timber shoring system that supports the sides of an excavation and which is designed to prevent cave-ins.

Short-term exposure a period of time ≤ 24 h that an excavation is open.

Slickensides surfaces with a soil mass which have been smoothed and striated by shear movements on these surfaces.

Sloping (sloping system) a method of protecting employees from cave-ins by excavating to form sides of an excavation that are inclined away from the excavation so as to prevent cave-ins. The angle of incline required to prevent a cave-in varies with differences in such factors as the soil type, environmental conditions of exposure, and application of surcharge loads.

Stable rock natural solid mineral material that can be excavated with vertical sides and will remain intact while exposed. Unstable rock is considered to be stable when the rock material on the side or sides of the excavation is secured against caving-in or movement by rock bolts or by another protective system that has been designed by a registered professional engineer.

Standard penetration test (SPT) the number of blows required to drive a split spoon sampler a distance of 12 in after an initial penetration of 6 in is referred to as an *N* value or SPT *N* value. This results from counting the number of blows of a 140 lb hammer, falling 30 in, required to advance a 2 in outer diameter split-barrel sampler 12 in through a soil mass.

Structural ramp a ramp built of steel or wood, usually used for vehicle access. Ramps made of soil or rocks are not considered structural ramps.

Submerged soil soil which is underwater or is free seeping.

Support system a structure such as underpinning, bracing, or shoring, which provides support to an adjacent structure, underground installation, or the sides of an excavation.

Tabulated data tables and charts approved by a registered professional engineer and used to design and construct a protective system.

Trench box see "Shield."

Trench (trench excavation) a narrow excavation (in relation to its length) made below the surface of the ground. In general, the depth is greater than the width, but the width of a trench (measured at the bottom) is not greater than 15 ft (4.6 m). If forms or other structures are installed or constructed in an excavation so as to reduce the dimension measured from the forms or structure to the side of the excavation to 15 ft (4.6 m) or less (measured at the bottom of the excavation), the excavation is also considered to be a trench.

Total stress At a given point in a soil mass the sum of the net stress across contact points of soil particles (effective stress) plus the pore-water pressure at the point.

Unconfined compressive strength the load per unit area at which a soil will fail in compression. It can be determined by laboratory testing, or estimated in the field using a pocket penetrometer, by thumb penetration tests, and other methods.

Under-consolidation the condition that exists if a soil deposit is not fully consolidated under the existing overburden pressure and excess hydrostatic pore pressures exist within the material.

Uprights the vertical members of a trench shoring system placed in contact with the earth and usually positioned so that individual members do not contact each other. Uprights placed so that individual members are closely spaced, in contact with or interconnected to each other, are often called "sheeting."

Wales horizontal members of a shoring system placed parallel to the excavation face whose sides bear against the vertical members of the shoring system or earth.

Wet soil soil that contains significantly more moisture than moist soil, but in such a range of values that cohesive material will slump or begin to flow when vibrated. Granular material that would exhibit cohesive properties when moist will lose those cohesive properties when wet.

References

Lindeburg, M.R. (1990) *Civil Engineering Reference Manual*, 4th edn, Professional Publications, Belmont, CA.

OSHA (Occupational Safety and Health Administration) (2012) 29 CFR 1926. *Construction*.

Ricketts, J.T., Loftin, M.K., and Merritt, F.S. (eds) (2003) *Standard Handbook for Civil Engineers*, 5th edn, McGraw-Hill, New York.